PUNJAB
IRRIGATION RESEARCH INSTITUTE

Director
Dr F. McKENZIE TAYLOR, M.B.E., Ph.D., D.Sc., F.I.C.

REPORT
FOR THE YEAR ENDING
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PUNJAB IRRIGATION RESEARCH INSTITUTE STAFF

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Head of Section ..	J. K. Malhotra, M.A., Ph.D. (On Special Duty, Sind-Punjab Dispute). Mohan Lal Madan, M.A. (Offg., in addition).
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(continued overleaf)

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INTRODUCTION

The Hydraulic Section has been kept fully occupied during the year and considerable extensions of the river research station have been necessary. The river models under examination during the year have included Jumna above Tajewala Headworks in connection with the control of shingle entering the canal; the control works on the river Sutlej above Suleimanke and Islam Headworks; the river Indus in connection with the construction of Kalabagh weir and the river Ravi for silt control at the Headworks at Madhopur. Recommendations made as a result of some of these investigations have already been put into operation.

A large number of models of rapids and falls have been examined with the object of controlling either side or bed scour. The suggestions made have been adopted in many cases.

A lengthy experiment on the design of silt extractors has been in progress. This experiment has particular reference to silt extractors in lined channels.

The Experimental Station established last year for the examination of different forms of outlets has continued to operate. A. P.M.'s have been under investigation.

The most important work with the Physics Section has been the investigation of the development of negative pressure in soils of high moisture content and the application of the results to the measurement of the depth of water-table. The main factor determining the negative pressure developed is the grade of the material. Owing to the development of the negative pressure, variations in well levels in high water-table areas may not indicate movements of the free water-table. The effect of grade of material on the negative pressure developed may account for the apparent variations in water-table depth at points close together, since differences in the mechanical composition of the soil will be reflected in the readings of water depths in the wells.

An investigation of the application of the electrical conductivity method for the detection of cavities under a weir was carried out at Balloki. The results obtained, when compared with actual quantities of sand used in grouting, show that unsound portions were fairly accurately detected.

The Mathematical Section has been studying bed silt movement in channels. The slope-discharge formula developed in earlier work held good when bed silt movement was small. When the quantity of bed silt moved is large, the slope is considerably steeper than the regime slope given by the formula. Experiments have been in prog ress to develop a method of measuring bed silt charge.

The work done is described under the following Headings :—

A—River Models.

B—Models of Rapids and Falls.

C—Silt Experiments.

D—Seepage and Regeneration Experiments.

E—Outlet Experiments.

A—RIVER MODELS.

Work on the following River Models was carried out :—

I—Further experiments on River Diversion on a model of the River Indus above and below *Kalabagh Headworks*.

II—River Diversion experiments on a model of the River Sutlej above *Suleimanke Headworks*.

III—Bank protection and River Training experiments on a model of the *Panjnad Headworks*.

IV—A study of a model of the river above Panjnad Headworks to determine the effect of scraping the bela upstream of the left divide wall on the approach to the left pocket.

V—Training of the River Ravi above the *Madhopur Pocket*.

VI—An investigation of a model of River Sutlej above *Islam Headworks* for determining methods to protect the Left retired embankment.

VII—An investigation of a model of the River Jumna above *Tajewala Headworks*.

I—Further experiments on River Diversion on a model of the River Indus above and below Kalabagh Headworks.

An account of the experiments carried out with three different schemes of river diversion examined on the model of Kalabagh Headworks was given in the last year's annual report.

The Chief Engineer ordered that diversion tests should again be carried out after moulding the model to the detailed and latest river survey to be supplied by the Executive Engineer, Kalabagh. The river tray downstream of the weir was extended by 150 feet in order to allow sufficient room for the development of the downstream leading cut. The precautions taken in this case were : (1) The grade of the shingle used for moulding the bed of the river was such that no shingle movement took place with a discharge equivalent to 30,000 cusecs. (2) The diversion cuts were made in pure screened sand. These precautions met the requirements of the Superintending Engineer, Upper Jhelum Canal, mentioned in his note dated the 3rd April, 1911.

The diversion cuts were made according to the alignment shown on the plan signed by the Superintending Engineer, Upper Jhelum Canal, on the 11th March 1911.

Testing the correctness of the model—Before starting with the diversion experiments the model was run with a discharge equivalent

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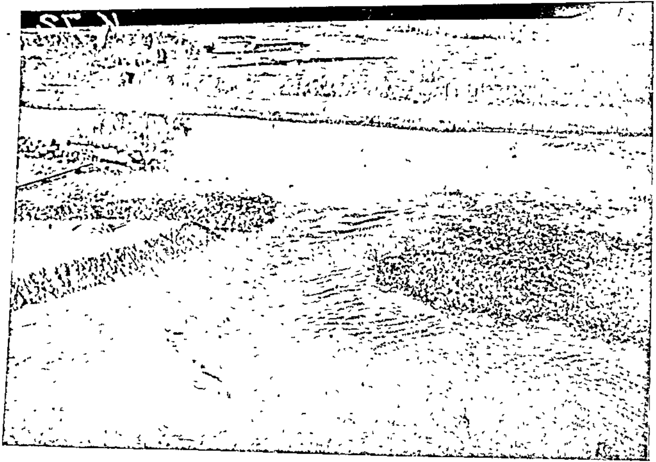
Testing the correctness of the model—Before starting with the diversion experiments the model was run with a discharge equivalent

Fig. 1

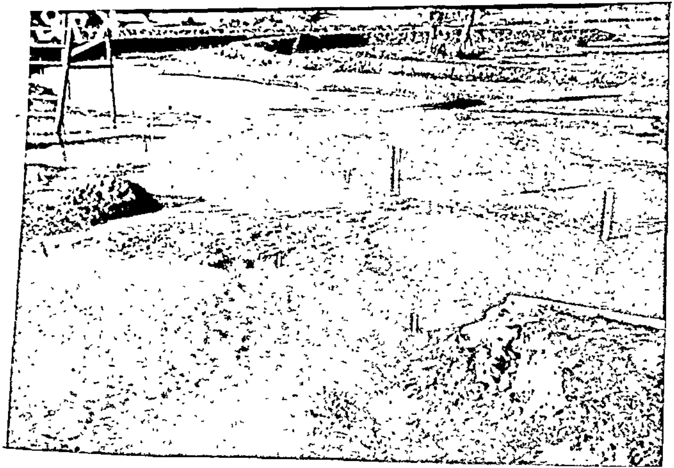
KALABAGH MODEL



Conditions of flow in the right creek before diversion

FIG. 2

KALABAGH·MODEL



Conditions of flow in the Central Creek before diversion

FIG. 3

KALABAGH HEADWORKS

PLAN SHOWING POSITIONS OF PROPOSED DIVERSION CUTS.

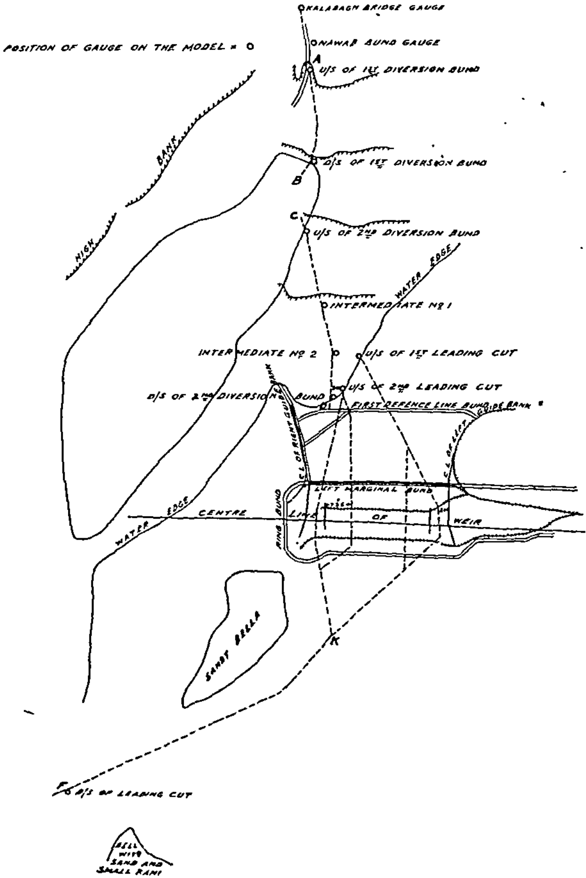
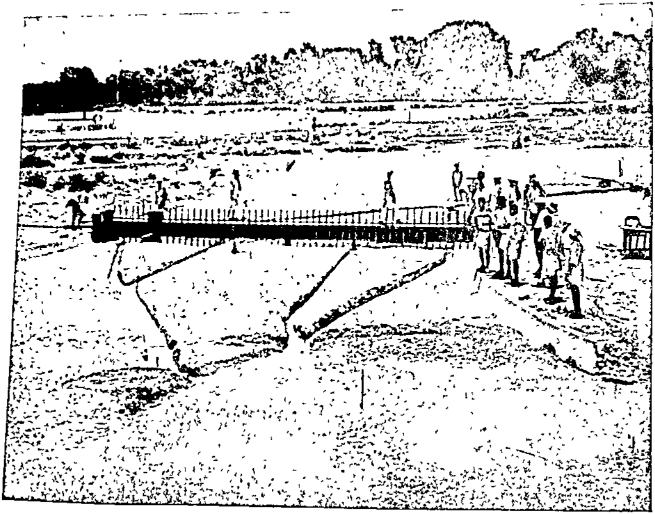


FIG. 4

KALABAGH MODEL



Conditions of the cuts at partial bunding of right creek

FIG. 5

KALABAGH MODEL

Conditions of the cuts at complete bunding of right creek

to 31,000 cusecs, and water levels at points corresponding to those taken on the prototype were observed in the right as well as in the central channel. These observations were compared with the prototype figures and are shown in Tables I and II. From an examination of these tables it will be seen that the water surface profile in the right channel is exactly reproduced on the model. At certain points in the central channel, however, there was some difference between the water levels observed on the model and the prototype figures given for the corresponding points. This was explained by the Overseer, Kalabagh, to be due to the existence of a small rapid at the right edge of the central channel which was not shown on the plan. This was made and the differences in the water level were adjusted to some extent.

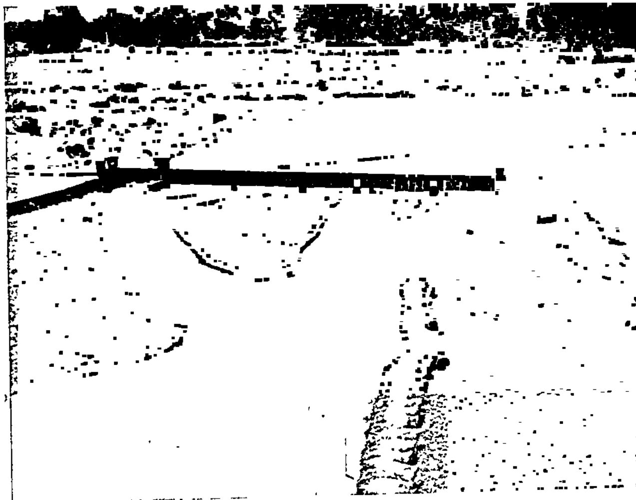
The Executive Engineer, Kalabagh, examined the model before the commencement of the river diversion. Along the left bank of the central channel above the heads of the leading cuts there was just a trickle of water flowing. The gauges at this point were, however, correct. According to the bed levels, the depth of water at this point was only ·68 foot corresponding to ·04 foot or ½″ on the model. The Executive Engineer, from his knowledge of the actual conditions, was of the opinion that more water should be passing along that side. Some shingle was, therefore, dropped just below the rapid as desired by him to allow more water to flow along the left bank of the central channel. This had the desired effect of passing more water along the left bank of the central channel, and raising the gauge at the head of the left side leading cut. The conditions of flow in the right and the central channels before the commencement of the diversion are illustrated in Figs. 1 and 2. Eleven gauges at points desired by the Superintending Engineer were erected for recording the water levels. The position of the gauges is shown in Fig. 3.

Closing the Right Channel—The cuts were opened at 12-20 p.m. and the closing of the central channel was commenced. It was suggested by the Executive Engineer, Kalabagh, who was present at this stage of the experiments, that the channel should be closed by dropping in stones instead of sand bags. This was done accordingly and the whole operation took six hours. The development of the cuts at different stages of the diversion of the right channel are illustrated in Figs. 4 and 5.

The gauges were observed after every fifteen minutes. A record of the observations is shown in Table III. An attempt was also made to measure the discharges in the central and the right channels. It was, however, not possible to take any accurate measurements. A ratio of the discharges in the two channels is given in Table IV. In addition, the velocity of flow of water in the cuts and the development of the cuts at different stages was noted. This is given in Table V. The gauges recorded at the time when the right channel was completely closed show that the maximum rise of water at the

FIG. 7

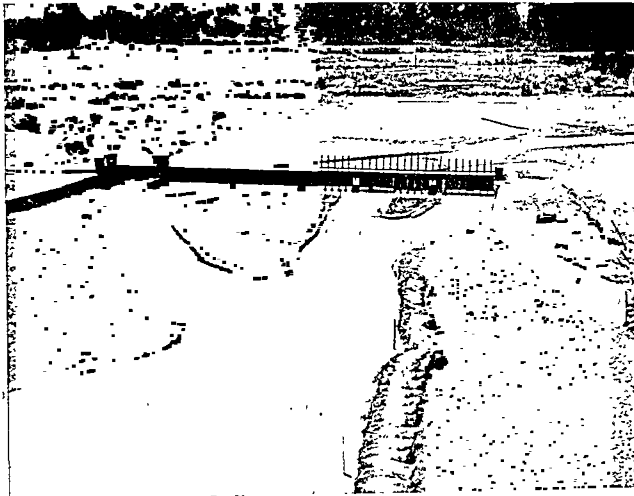
KALABAGH MODEL



Conditions of cuts after complete bunding of central creek

FIG. 7

KALABAGH MODEL



Conditions of cuts after complete bunding of central creek

FIG. 8

KALABAGH MODEL

Conditions of the cuts and bund-

FIG. 9

KALABAGH MODEL

Conditions of the cuts after complete bunding of central creek

Dimensions of the cuts, in feet—

Right main cut	900
Right subsidiary cut	450
Left subsidiary cut	420
Left main cut	700

It will be seen from the above table and Fig. 9 that channels were formed in front of the whole of the weir excepting a bela which existed in front of the central portion. The following conclusions were obtained from the series of experiments.

The cuts according to the alignment proposed by the Superintending Engineer, Upper Jhelum Canal, have been examined on a model of the Kalabagh Headworks moulded to the latest survey. It has been shown that—

(1) The diversion of the right channel causes a heading up of about ·5' at the gauges erected at the heads of the leading cuts. Most of the water diverted from the right channel passes to the central channel without affecting greatly the development of the cuts.

(2) Closing of the central channel produces a marked development of the leading cuts.

(3) There is a heading up of about 4·5 feet at the gauges constructed at the heads of the cuts. The R. L. of the gauge is about 689·3.

It may be pointed out that the water level after the completion of the previous diversion test was 691·0. After the completion of the above experiments the Chief Engineer ordered that the following further schemes of diversion should be examined :—

(1) Closing the central channel (from the left flank) first, followed by the right channel.

(2) Closing the right channel first as in the original test with the diversion cuts of twice the width of those used in the first test.

(3) Making the diversion cuts to the dimensions already used and running the model for one monsoon before starting the diversion to see whether the cuts will develop or silt up.

The tests were made accordingly and are described below :—

1. *Closing the central channel (from the left flank) first, followed by the right channel*

The model was moulded according to the latest survey obtained from the Executive-Engineer, Kalabagh Headworks. The alignment of the cuts used for the diversion was the same as adopted before.

The diversion was effected with a discharge of 31,000 cusecs. The following order of closing the channels was adopted :—

(1) The central channel was closed first. The closing of the channel was carried out from the left flank in six instalments. The total period taken on the model for closing the channel was six hours which is equivalent to one month on the prototype.

(2) After completely closing the central channel the right channel was closed. The total period taken for closing this channel was also six hours.

Gauges were erected at the same points as in the previous test for recording the river levels. The positions of these gauges is shown in Fig. 3. Water levels were observed every fifteen minutes. Photographs illustrating the conditions of flow at different stages of the development of the cuts were also taken and are given in Figs. 10 to 20. Notes of important conditions of flow were taken during the operation of the model.

Conditions of flow at the head of the cuts before opening the cuts—The conditions of flow at the head of the cuts and at the Right Guide Bank in a discharge of 31,000 cusecs before opening the cuts are illustrated in Fig. 10. From an examination of this photograph it will be seen that the flow along the left flank of the central channel mainly takes place over the rapid in front of the cut. Very little flow occurs at the head of the second cut.

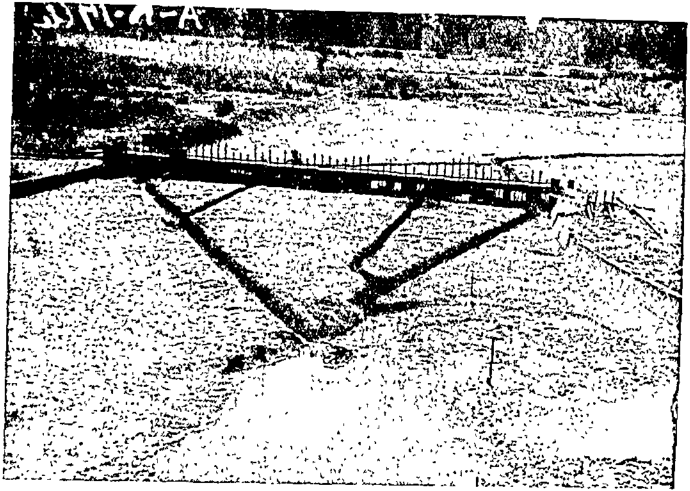
Conditions of flow after opening the cuts and before the commencement of river diversion—The bunds at the head of the cuts were next removed. When the cuts were opened, water flowing along the left bank of the central channel started going into the cuts. The conditions of flow when the water just entered the cuts without putting in the diversion bund are illustrated in Fig. 11. The area along the left flank downstream of the head of the right cut became dry. This is also shown in Fig. 11.

When the bunds in front of the head of the cuts are removed and the cuts allowed to flow, the gauges at the head of the cuts drop due to the draw of water in the cuts. The gauge fixed at the head of the 2nd or the right cut is affected accordingly.

Closing of the Central Channel—First Bunding up—The conditions of flow with the first portion of the bund in the central channel are shown in Fig. 12. The first instalment of the bund in the central channel did not affect the [illegible] the cuts, as no flow of water took [illegible] The left cut took practically [illegible] along the left flank of the central channel at the head of the cuts. The gauge statement after every fifteen minutes for the complete run is given in Table VI. As the gauges at the head of the right and the left cuts are the most important from the point of view of determining the heading up, the water-levels for these two points at each bunding are also given separately.

Fig. 10

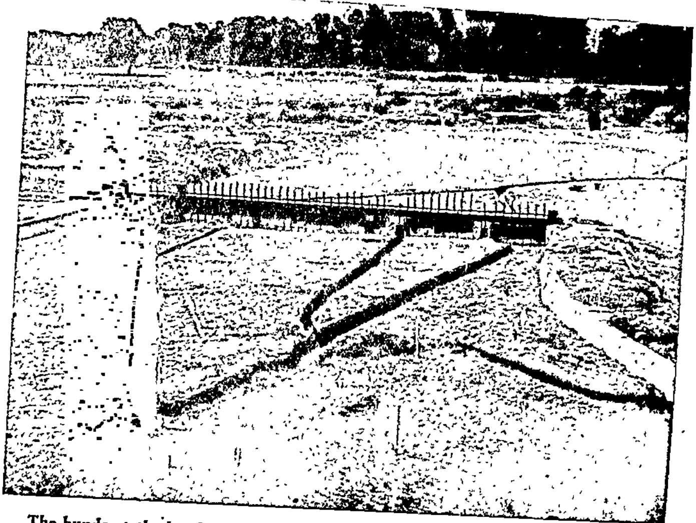
KALABAGH MODEL



Discharge in the river 31,000 cusecs

FIG. 11

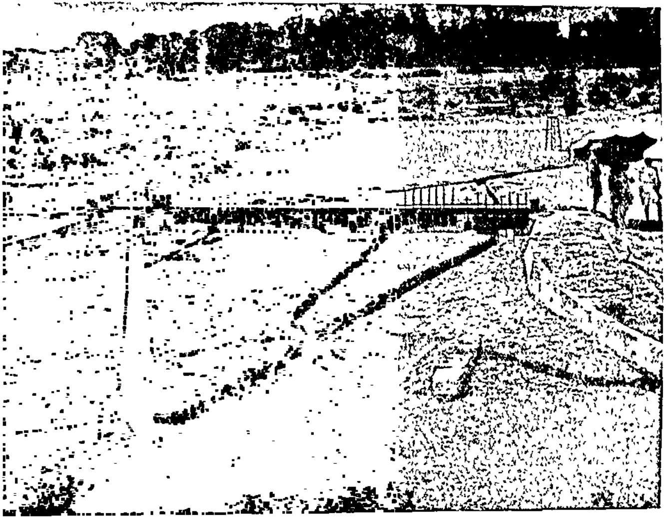
MODEL OF RIVER INDUS AND KALABAGH HEAD WORKS



The bunds at the head of the cuts removed and water allowed to enter the cuts

FIG. 12

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



First portion of the diversion bund constructed in the Central Channel

Third Bunding up—With the third bunding up the cuts started developing. The downstream cut also widened out. The right main cut became wide. The gauges are given below :—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After the 3rd bunding up of central channel	685·47	684·82
After the 3rd bunding up of right channel	84·92	81·07

REMARKS—Both the gauges are higher than those recorded in the previous test.

Fourth Bunding up of the central channel—With the fourth instalment both the right and the left cuts developed rapidly. The right cut at the head widened. The downstream cuts also developed. The conditions of flow are illustrated in Fig. 14. The gauges obtained are compared below :—

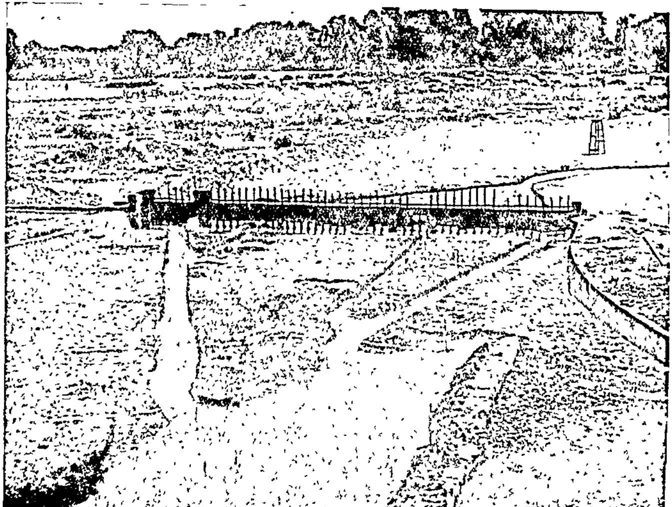
Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After fourth bunding up of the central channel	686·0	685·68
After the fourth bunding up of right channel	85·14	81·39

REMARKS.—The gauges at the head of the left and the right cuts in these tests are higher than those obtained in the previous tests

Fifth Bunding up of the central channel—With the fifth bunding up the island between the main and the subsidiary cuts was attacked and the left cut widened. Similarly the right cut also developed.

FIG. 14

MODEL OF INDUS AND KALABAGH HEADWORKS



Fourth bunding up of the Central Channel.

as used in the last test, i.e., by means of adding layers of boulders all along the bund.

First Bunding up of the right channel (central channel closed)—The conditions with the first instalment of the bund in the right channel are illustrated in Fig. 16. The closing of the right channel affected very much the development of the right cut. The island between the main and the subsidiary left cut was altogether washed away. The subsidiary cut on the right side developed very much. The downstream cuts also developed. The gauges are given below :—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After the first bunding up of the right channel	687·71	687·89
After the first bunding up of the central channel	85·14	85·35

REMARKS—The gauges when the central channel was closed first are higher than those obtained when the right channel is closed first.

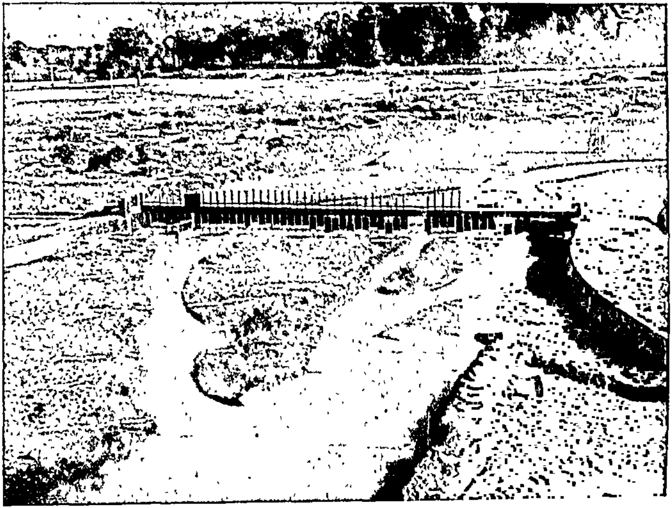
Second Bunding up of the right channel (central channel closed)—The conditions with the second bunding up of the right channel are illustrated in Fig. 17. The right cut considerably developed. A portion of the island between the right and the left cuts was washed away. The gauges are given below :—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After the second bunding up of the right channel	687·82	688·04
After the second bunding up of the central channel	85·57	85·74

REMARKS—The gauges in this case are also higher than those obtained in the previous one.

FIG. 16

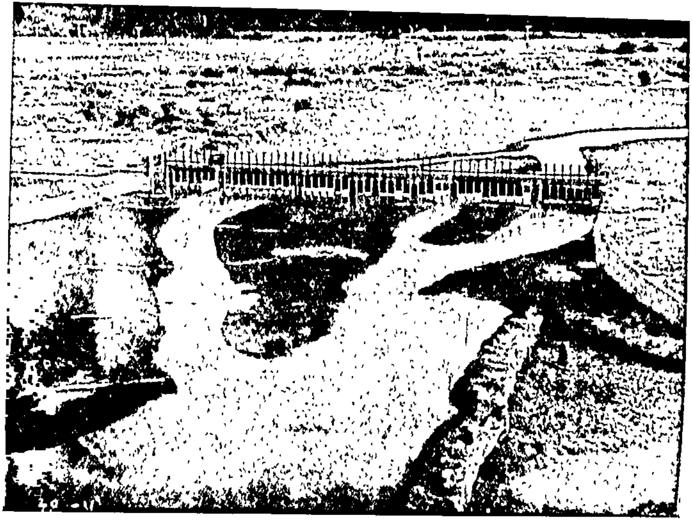
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of flow when first portion of the bund in the right channel has been *constructed*. (The central channel having been closed first.)

FIG. 17

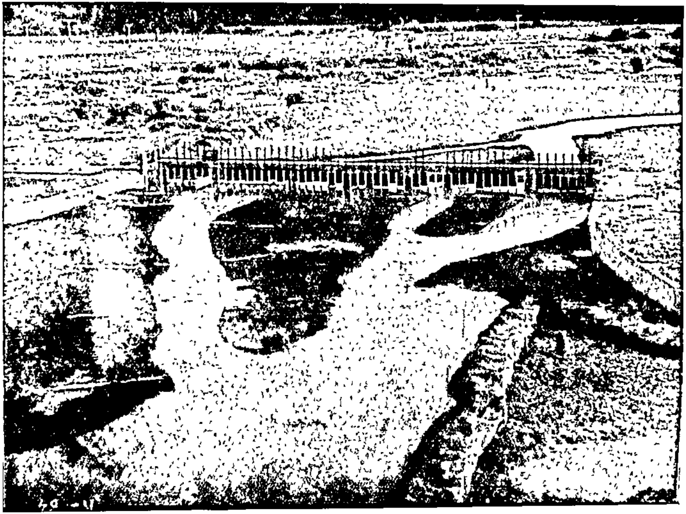
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of flow when second portion of the bund in the right channel has been constructed (The central channel having been closed first).

FIG. 17

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of flow when second portion of the bund in the right channel has been constructed (The central channel having been closed first).

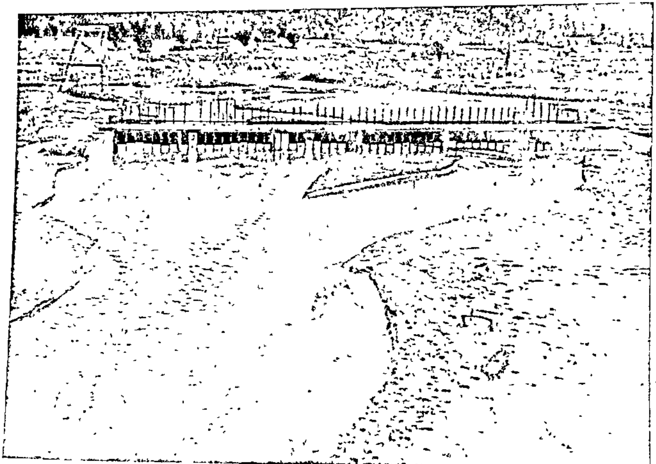
FIG. 18

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

Conditions of flow upstream of the weir when the whole of the river is diverted on to the weir

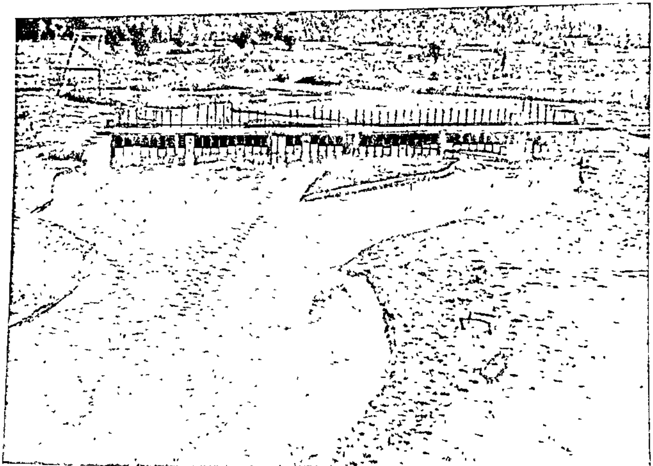
FIG. 19-A

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS AFTER COMPLETE DIVERSION



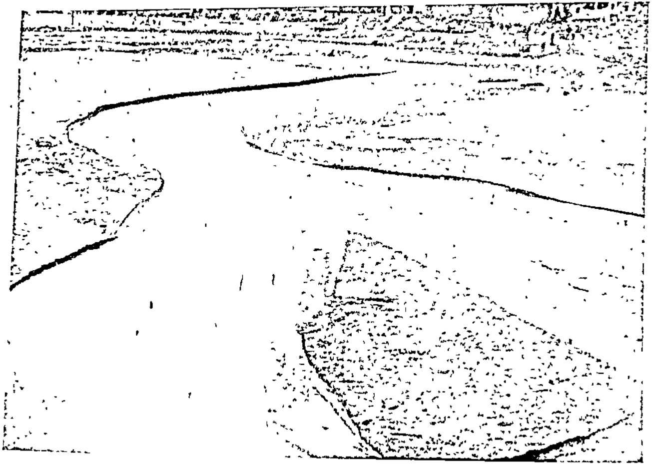
Conditions of flow downstream of the weir looking upstream when the whole of the river is diverted on to the weir. The cut downstream of the weir developed completely at the end of the run

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS AFTER COMPLETE DIVERSION



Conditions of flow downstream of the weir looking upstream when the whole of the river is diverted on to the weir. The cut downstream of the weir developed completely at the end of the run

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of flow in the cut downstream of the weir looking downstream. The cut downstream of the weir developed completely at the end of the run.

FIG. 19-B

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

Conditions of flow in the cut downstream of the weir looking downstream. The cut downstream of the weir developed completely at the end of the run.

(4) The gauges fixed at the head of the right and the left leading cuts record higher levels when the central channel is closed first than those obtained when the right channel is closed first. The difference in these gauges is about one foot. When the central channel is closed first the highest level obtained is R. L. 90·2 and when the right channel is closed first it is R. L. 89·2.

A rise of about 5·5 feet occurs on the gauges fixed at the head of the left cut when the central channel is closed first. When the right channel is closed first a rise of 4·5 feet takes place. This is against an expected rise of 2 feet.

Summary and conclusions

A model of the river Indus above and below the Kalabagh Headworks has been examined to determine the most suitable methods of river diversion. It has been shown that higher gauges are obtained close to the headworks when the central channel of the river is closed first.

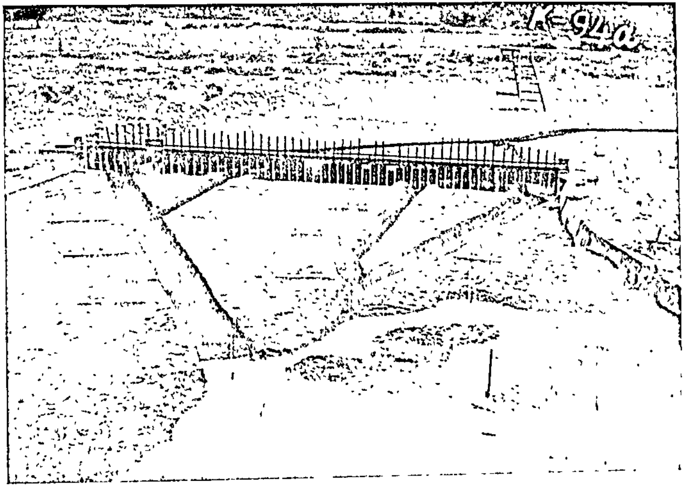
2. *Closing the right channel first, as in the original test, with the diversion cuts twice* [illegible] *the first test*—The model was remoulded [illegible] er survey and all the gauges were erect[illegible] places. The cuts were made according t[illegible] used in the previous tests. The width of the cuts adopted in this test was double of that used in the previous tests. The cuts made are shown in Fig. 21. The order of closing the channels was the same as that used in the last test, i.e., the central channel was closed first and the right channel later. Before the construction of the diversion bund the cuts were opened and water was allowed to flow into them. The conditions of flow after removing the bunds are illustrated in Fig. 22. An examination of this figure shows that when the bunds at the heads of the cuts are removed, the left cut draws practically the whole of the discharge flowing along the left flank. The right cut remained almost dry.

Bunding up of the central channel—The central channel was closed in six instalments. The whole operation took six hours as before.

First Bunding up—The conditions of flow after constructing the first portion of the bund in the central channel are illustrated in Fig. 23. It will be seen from this figure that the left cut took the whole of the discharge and there was little flow taking place into the right cut. The conditions in the main cut downstream of the weir are shown in Fig. 24. A comparative statement of the gauges at the heads of the right and the left cuts is given below. The corresponding gauges for both the previous tests, i.e. when the central channel is

FIG. 21

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts

(4) The gauges fixed at the head of the right and the left leading cuts record higher levels when the central channel is closed first than those obtained when the right channel is closed first. The difference in these gauges is about one foot. When the central channel is closed first the highest level obtained is R. L. 90·2 and when the right channel is closed first it is R. L. 89·2.

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2. *Closing the right channel first, as in the original test, with the diversion cuts twice the width of those used in the first test*—The model was remoulded according to the latest river survey and all the gauges were erected at their corresponding places. The cuts were made according to the alignment used in the previous tests. The width of the cuts adopted in this test was double of that used in the previous tests. The cuts made are shown in Fig. 21. The order of closing the channels was the same as that used in the last test, i.e., the central channel was closed first and the right channel later. Before the construction of the diversion bund the cuts were opened and water was allowed to flow into them. The conditions of flow after removing the bunds are illustrated in Fig. 22. An examination of this figure shows that when the bunds at the heads of the cuts are removed, the left cut draws practically the whole of the discharge flowing along the left flank. The right cut remained almost dry.

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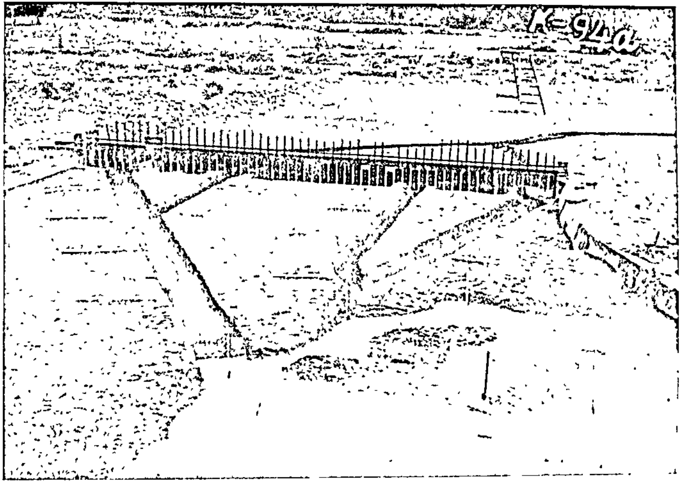
FIG. 21

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts

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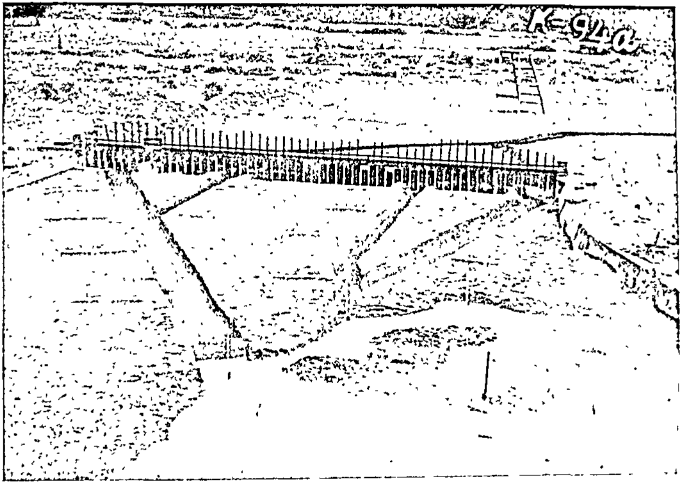
Fig. 21

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts

FIG. 22

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

Double width cuts (Bunds removed)

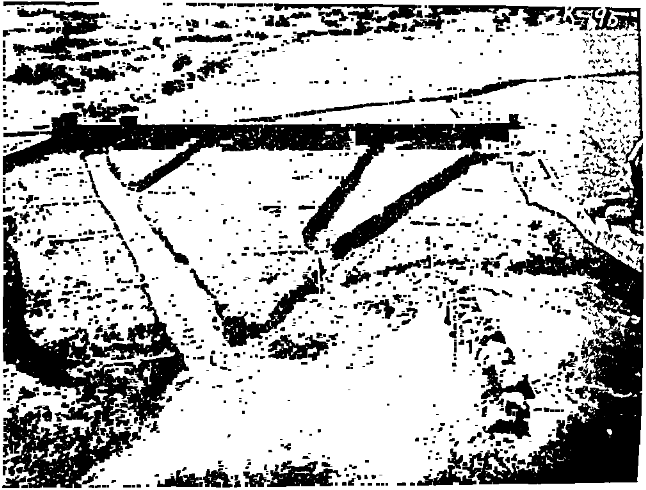
FIG. 22

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

Double width cuts (Bunds removed)

FIG. 23

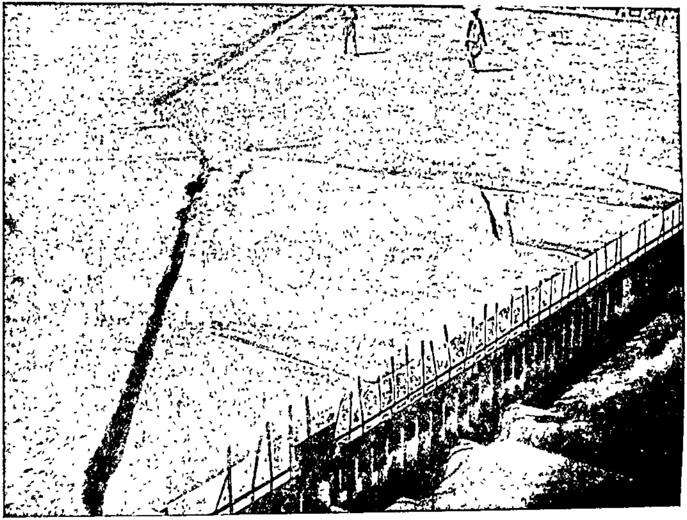
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts

FIG. 21

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
First bunding of central channel.
A view of the cuts downstream of the weir.

Third Bunding up—With the [illegible] sidiary and the main right cuts star[illegible] carried a major portion of the discharge. The conditions [illegible] in Fig. 26. The conditions of flow on the cut downstream of the weir are illustrated in Fig. 27. The gauges are given below :—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After third bunding up of the central channel (double width cuts).	684·34	681·39
After third bunding up of the central channel (normal width cuts.)	85·47	84·82
After third bunding up of the the right channel (normal width cuts).	84·92	81·07

REMARKS—The gauge at the head of the left cut with double width cuts is lowest.

Fourth Bunding up—The conditions of flow with the fourth bunding are illustrated in Fig. 28. The cut downstream of the weir widened. The gauges are given below :—

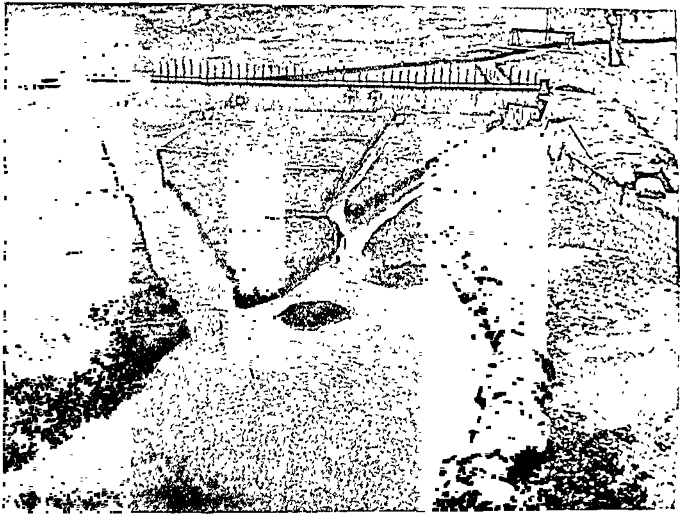
Gauge	U/S of left leading cut	U/S of right lead-cut
	R. L.	R. L.
After fourth bunding up of the central channel (double width cuts).	684·64	681·50
After fourth bunding up of central channel (normal width cuts).	86·0	85·68
After fourth bunding up of the right channel (normal width cuts).	85·14	81·39

REMARKS—The gauge at the head of the left cut with double width cuts is the lowest.

Sixth or complete Bunding up of the central channel—When the central channel was completely closed the conditions of flow obtained are illustrated in Fig. 29. It will be seen from this figure that the right cut became very wide at the head and a portion of the island between

Fig. 26

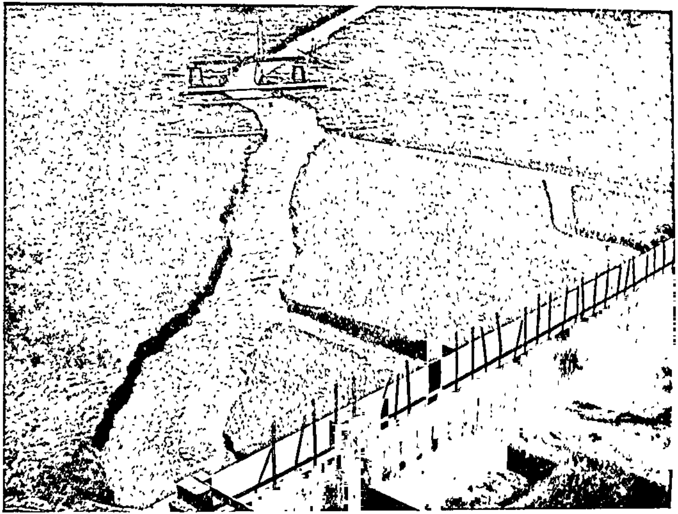
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
3rd bunding of central channel.

FIG. 27

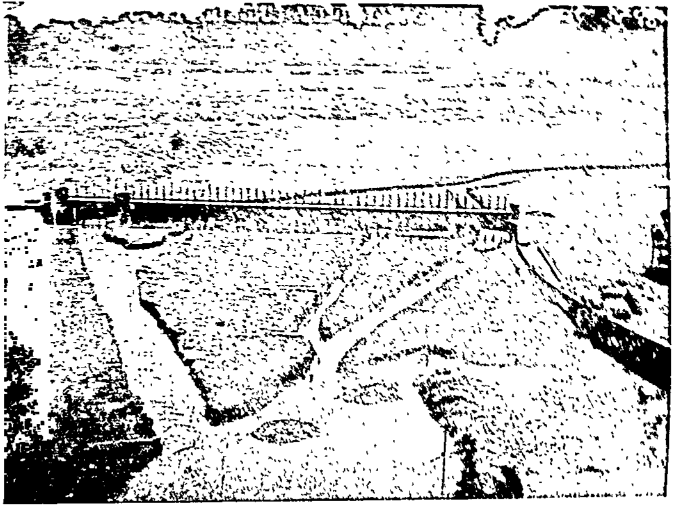
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
3rd bunding of central channel.
A view of the cuts downstream of the weir.

Pl. 28

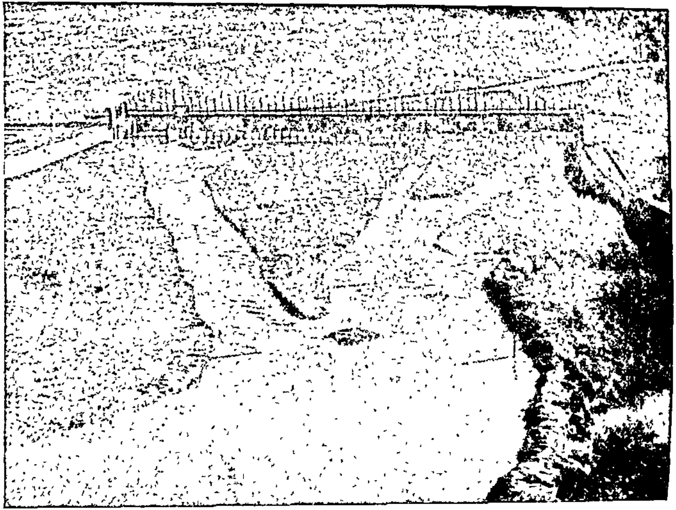
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
4th bunding of central channel.

FIG. 29

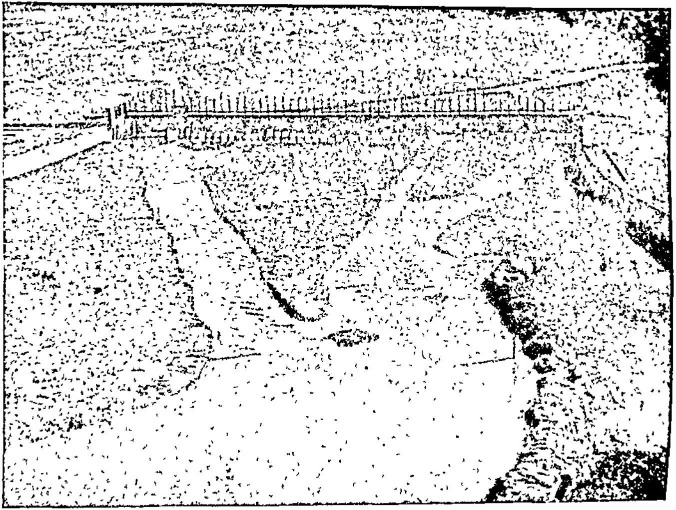
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
Complete bunding of central channel.

FIG. 29

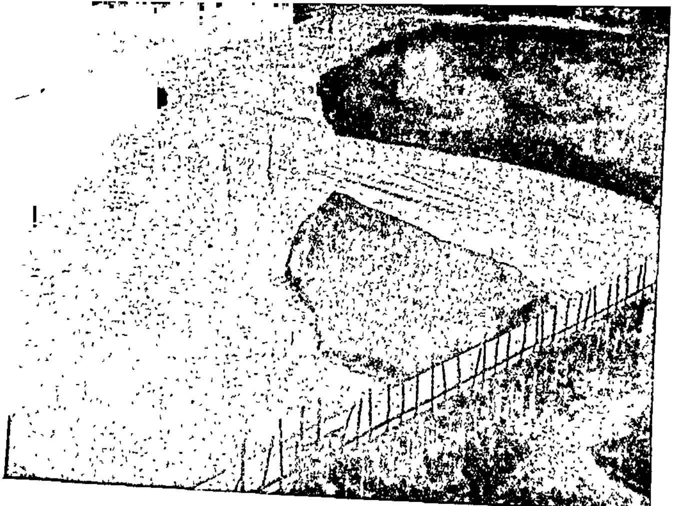
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
Complete bunding of central channel.

Fig. 30

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
Complete bunding of central channel.
A view of the cuts downstream of the ...

the subsidiary and main cuts was washed away. The velocity at the head of this cut was 7·5 feet per second and was the same as at the head of the left cut. The conditions of the cuts downstream of the weir after complete bunding up of the central channel are shown in Fig. 30. It will be seen from this figure that a rapid development took place of these cuts and a considerable area downstream of the weir was occupied by the channels. The gauges are given below :—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After complete bunding up of the central channel (double width cuts).	684·82	684·61
After complete bunding up of the central channel (normal width cuts.)	87·38	87·39
After complete bunding up of the right channel.	85·14	81·71

All the gauges obtained after closing the central channel first, with double width cuts, are compared with the corresponding gauges obtained when the central channel is closed first with normal width cuts. These are given below :—

Statement showing the gauges after closing only the central channel with double and normal width cuts

Position of the gauge	Gauge	
	With double width cuts	With normal width cuts
	R. L.	R. L.
At Kalabagh Bridge ..	688·0	687·9
At Nawab's bund ..	86·8	87·78
U/S of first diversion bund ..	86·65	87·65
D/S of first diversion bund ..	86·43	86·69
U/S of second diversion bund	85·25	87·83
Intermediate No. 1 ..	86·43	87·61
Intermediate No. 2 ..	86·43	87·71
D/S of second diversion bund..	84·93	87·61
U/S of left leading cut ..	84·82	87·38
U/S of right leading cut, ..	84·61	87·39
D/S of the leading cut ..	76·00	76·20

From an examination of the above table the following indications are obtained :—

(1) At all the gauge points, excepting the gauge constructed at the bridge site, the levels are lower in the case of the double width cuts.

(2) The gauges at the head of the cuts with the double width cuts after closing the central channel are about 2·5 feet lower than the corresponding gauges when the cuts are of normal width.

(3) There is practically no rise of water level at the gauge at the head of the left cut after closing the central channel. The gauge at the head of the right cut is higher than the initial level by one foot.

Closing the right channel of the rive[illegible]
channel—After the central channel was [illegible]
of the right channel was commenced. [illegible]
hours as before. The method adopted was also the same.

First bunding up of the right channel—With the first bunding up of the right channel, the right leading cut developed considerably and the nose of the island between the right and the left cuts was eroded. The gauges for this experiment as well as those obtained in the previous experiments are given below—

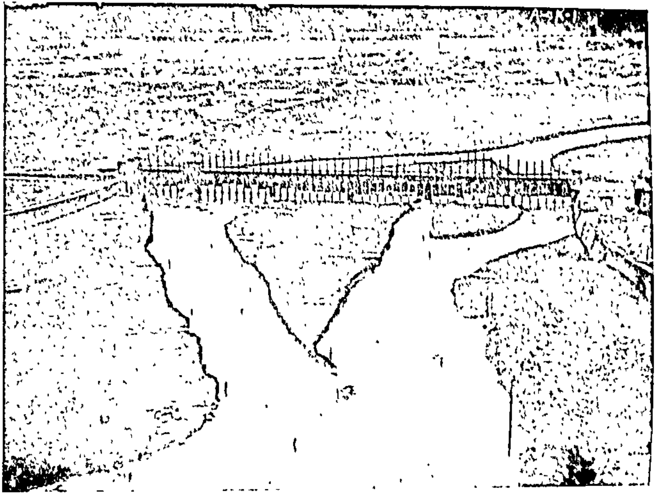
Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After first bunding up of the right channel, central channel having been closed (double width cuts).	685·68	685·46
After first bunding up of the right channel, central channel having been closed (normal width cuts).	87·71	87·89
After first bunding up of the central channel, the right channel having been closed (normal width cuts).	85·14	85·35

REMARKS—The gauges at the head of the cuts are considerably lower with the double width cuts than those obtained with the normal width cuts when the central channel is closed first.

Second bunding up—With the second bunding up the conditions of flow in the cuts are illustrated in Fig. 31. From an examination of this figure it will be seen that the *bela* between the main and the subsidiary cuts is washed away completely. The island between the main and the subsidiary right cuts is also washed away to a great

FIG. 31

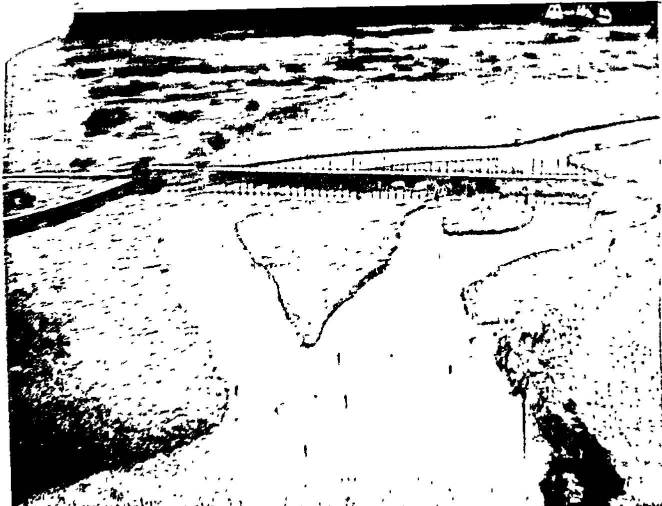
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.

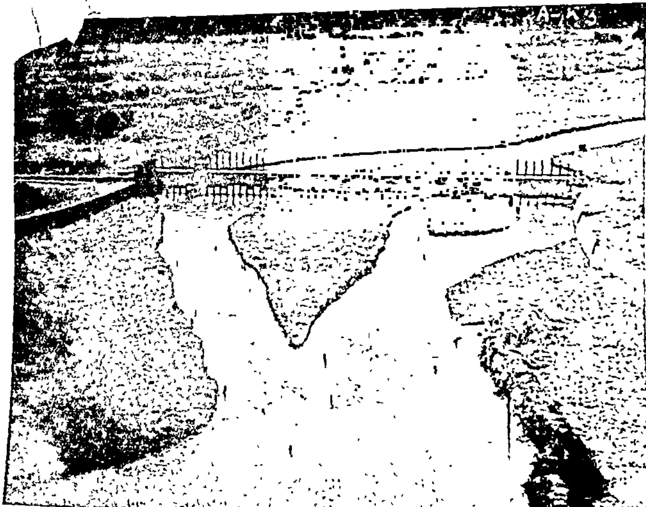
2nd bunding of the right channel. The central channel having been closed first

MODEL [illegible] AND KALABAGH HEADWORKS



Double width cuts.

MODEL OF RIVER INDUS AND KALAPAGH HEADWORKS



Double width cuts.
Conditions of flow of the 5th bunding of right channel.

Sixth or the complete bunding up of the right channel—The conditions of flow after the river was completely diverted by closing the right channel are illustrated in Fig. 34. An examination of this figure shows that, excepting for a small portion of the *bela* in front of the centre of the weir, the whole of the area was occupied by the channels. The channels also developed downstream of the weir. This is illustrated in Fig. 35. The gauges are given below—

Gauge	U/S of left leading cut	U/S of right leading cut
	R. L.	R. L.
After complete bunding up of the right channel, central channel having been closed (double width cuts).	686·75	686·32
After complete bunding up of the right channel, central channel having been closed (normal width cuts).	90·18	90·18
After complete bunding up of the central channel, the right channel having been closed (normal width cuts)	89·21	89·21

REMARKS—Same as in first bunding up.

From an examination of the foregoing results the indications obtained are as follows :—

(1) The double width cuts give the lowest levels. The heading up experienced at the head of the left leading cut is about 2·3 feet while at the head of the right leading cut it is 3·4 feet.

(2) When the central channel is closed first the final levels at the head of the leading cuts after complete diversion are somewhat higher than those obtained when the right channel is closed first.

(3) The double width cuts give the most satisfactory results from the point of view of heading up.

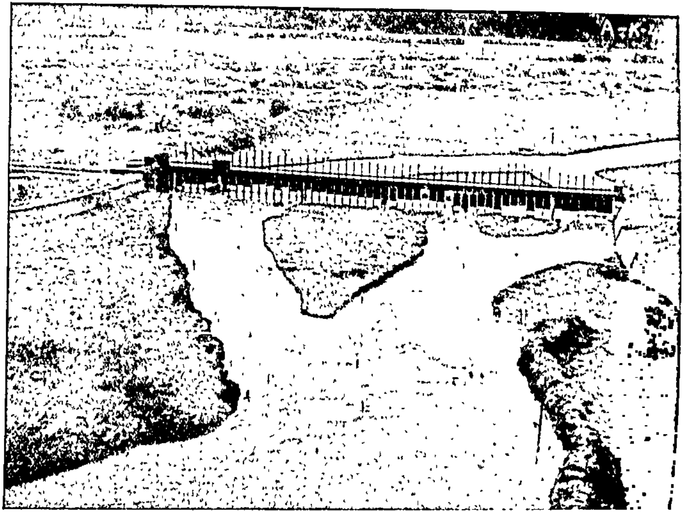
It is suggested that the left leading cut may be made double width while the right leading cut requires to be examined on the model.

3. *Making the diversion cuts to the dimensions already used and running the river for one monsoon season before starting the diversion to see whether the cuts will develop or silt up.*

The model was remoulded to the river survey used in the previous experiments and the cuts were made of the normal size as already used.

FIG. 31

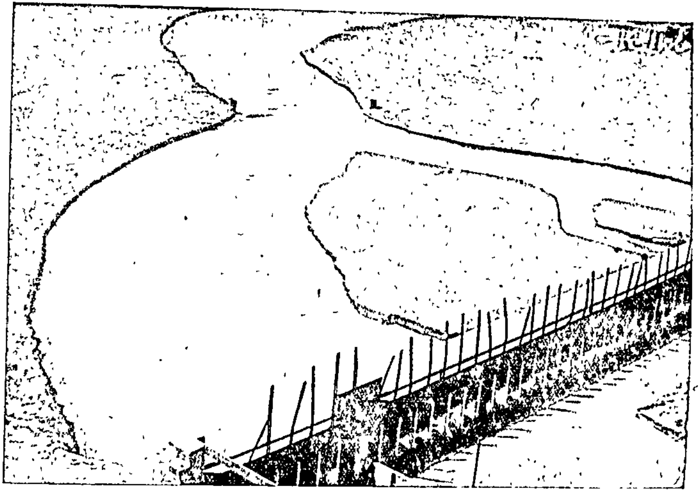
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
After complete divers on

FIG. 35

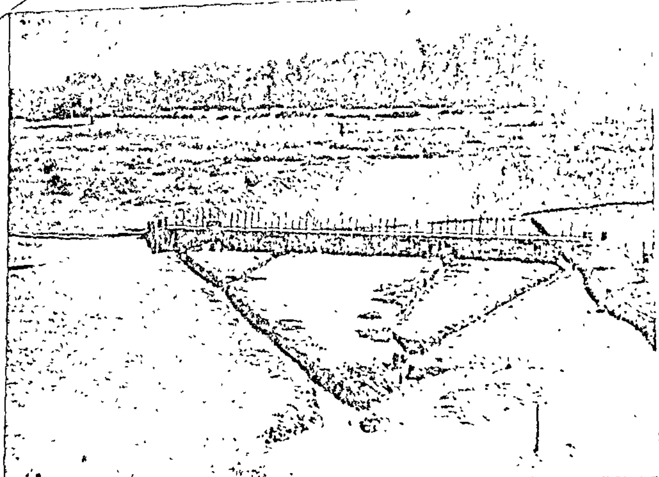
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Double width cuts.
Complete bunding of right channel.
A view of the cuts downstream of the weir.

Fig. 36

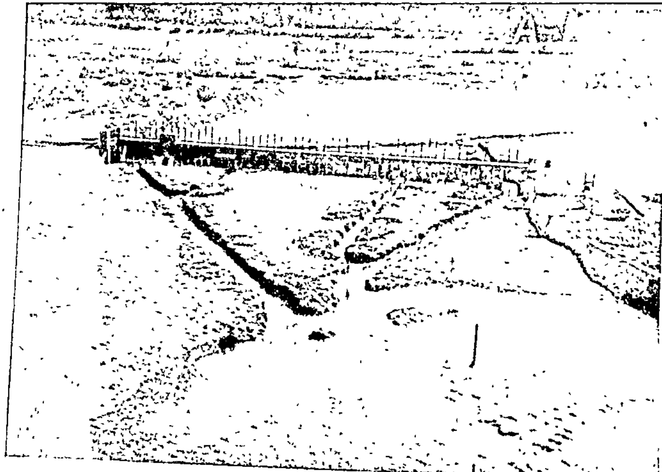
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Cuts running with a discharge of 29,000 cusecs.
Diversion bunds not constructed.

Fig. 37

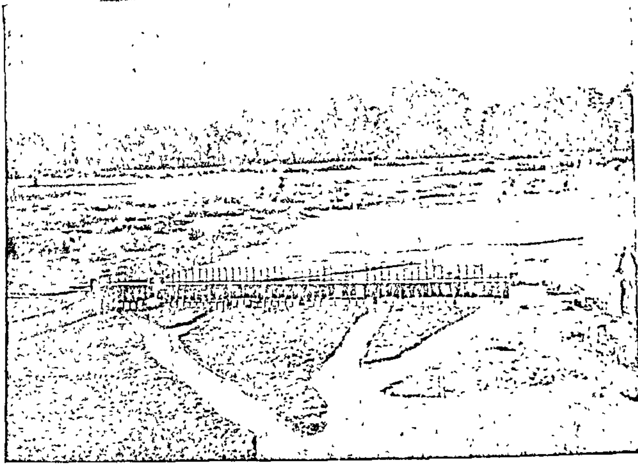
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Cuts running with a discharge of 47,000 cusecs.
Diversion bunds not constructed.

FIG. 38

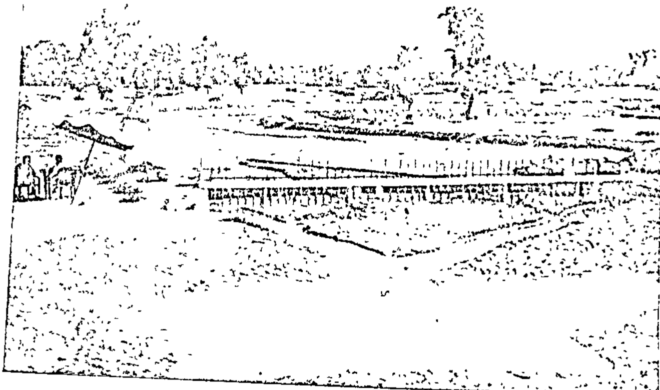
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Cuts running with a discharge of 55,000 cusecs.
Diversion bunds not constructed.

FIG. 39

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Cuts running with a discharge of 184,000 cusecs.
Diversion bunds not constructed.

FIG. 40

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

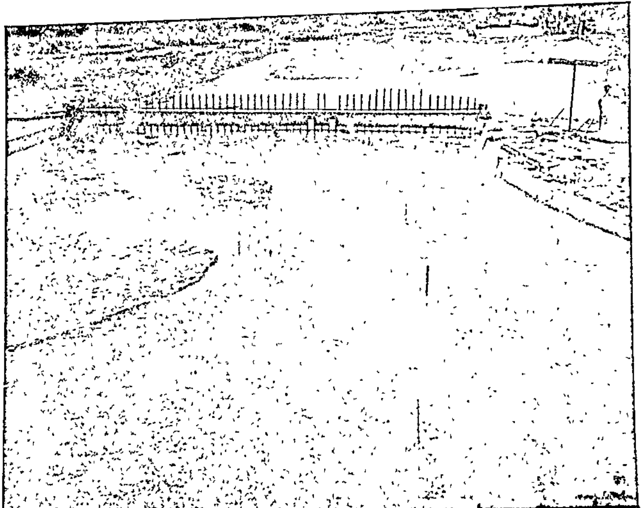
Cuts running with a discharge of 85,000 cusecs after the flood of 184,000 cusecs.
Diversion Bunds not constructed

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

Cuts running with a discharge of 85,000 cusecs after the flood of 184,000 cusecs.
Diversion Bunds not constructed

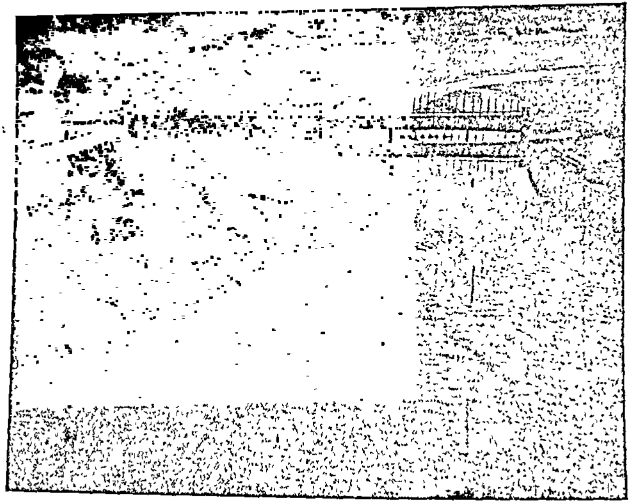
FIG. 41

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of flow with a discharge of 300,000 cusecs.
Diversion bunds not constructed.

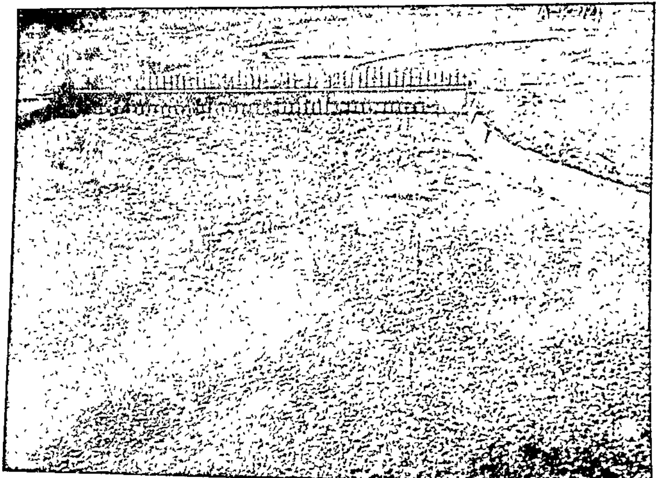
MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Cuts running with a discharge of 70,734 cusecs.
Diversion bunds not constructed

FIG. 43

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS



Conditions of the river bed upstream of the weir after one season run

FIG. 43

MODEL OF RIVER INDUS AND KALABAGH HEADWORKS

With a discharge of 23,721 cusecs in the river, water just started flowing into the left cut. With a discharge of 29,500 cusecs water flowed into both the cuts. The conditions of flow with this discharge are shown in Fig. 36. The conditions of flow with discharges of 47,000 cusecs and 55,000 cusecs are illustrated in Figs. 37 and 38, respectively.

After the above discharges had been run a flood discharge of 184,000 cusecs was run on the model to represent a winter freshet. The conditions of flow in the cuts downstream of the weir with this discharge are shown in Figs. 39 and 40. After running the freshet discharge the model was again run with a discharge equivalent to 70,000 cusecs. The right cut appeared to develop rapidly.

Flood discharge—A flood discharge equivalent to 300,000 cusecs was next run on the model. The conditions of flow with this discharge are illustrated in Fig. 41. From an examination of this figure it will be seen that all the *belas* on the right side appear to have been removed. The right half of the weir took an appreciable portion of the discharge.

After running the flood discharges, low discharges were run on the model. The conditions of flow with a discharge of 70,000 cusecs are illustrated in Fig. 42. From an examination of this figure it will be seen that the left cut does not take any water and has silted up. Water, however, flows through the right cut and over the *belas* on the right half of the weir. It indicates that silting has taken place in the falling flood. When the discharges in the river fell to 60,000 cusecs there was no water flowing against the weir. The conditions upstream of the weir in the dry are shown in Fig. 43. It will be seen from this figure that the left subsidiary and the main cuts silted up. The right cut also silted up but to a lower level than the left cut. A survey was taken of the bed and is plotted in Fig. 44.

From a study of these figures the following indications are obtained :—

(1) The cuts developed in the flood discharges but again silted up.

(2) The left portion of the weir silted up very heavily and no flow took place into the left cut with discharges in the river below 70,000 cusecs.

(3) The right cut also silted up but the bed was on a lower level than the left cut. No flow took place into these cuts with discharges lower than 60,000 cusecs.

Conclusion—When no diversion bunds are constructed in the river and the cuts are allowed to run for a period equivalent to one year the cuts silt up at the end of the flood season.

FIG. 15

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

A view of the completed model showing the position of bunds, etc.

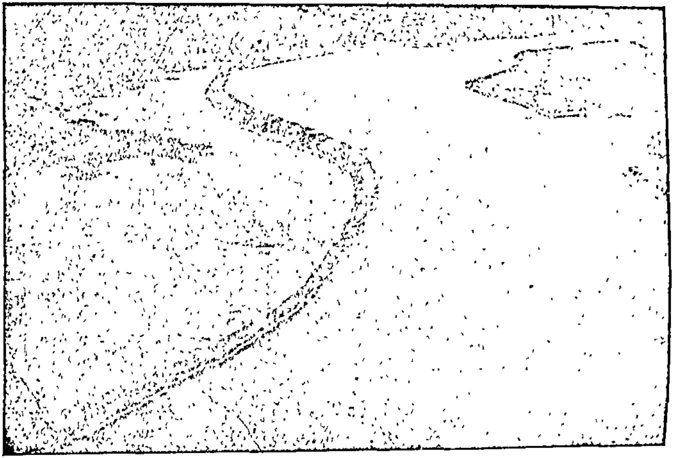
FIG. 46

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

Low discharge in the river

FIG. 47

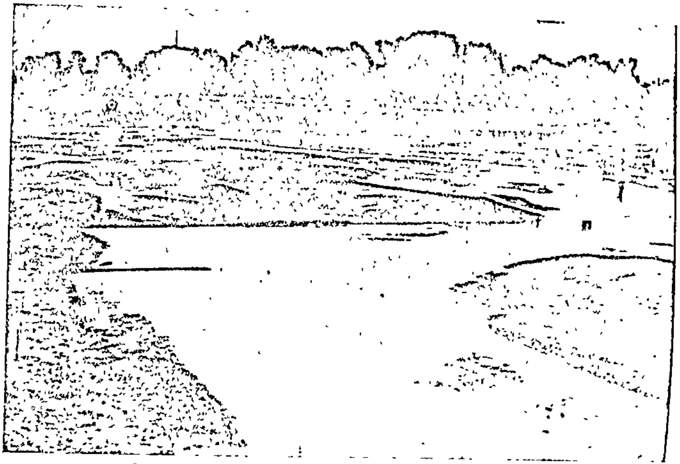
MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



This photograph shows the diversion bund between R. D. 63–59 just at the stage of breaching

FIG. 18

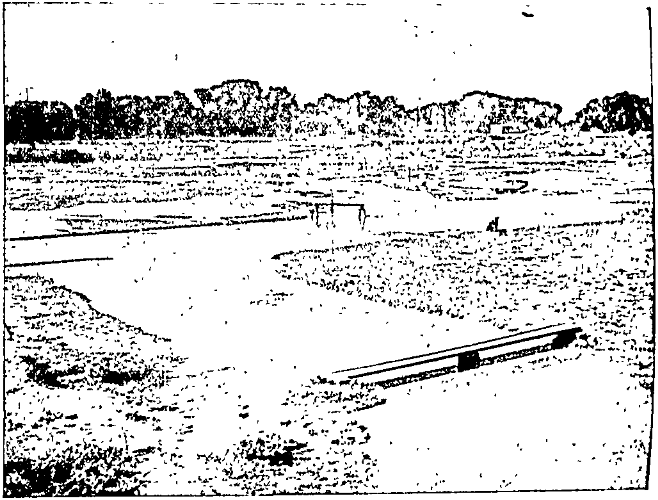
MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



Photograph showing the flow taking place along the main bund

FIG. 42

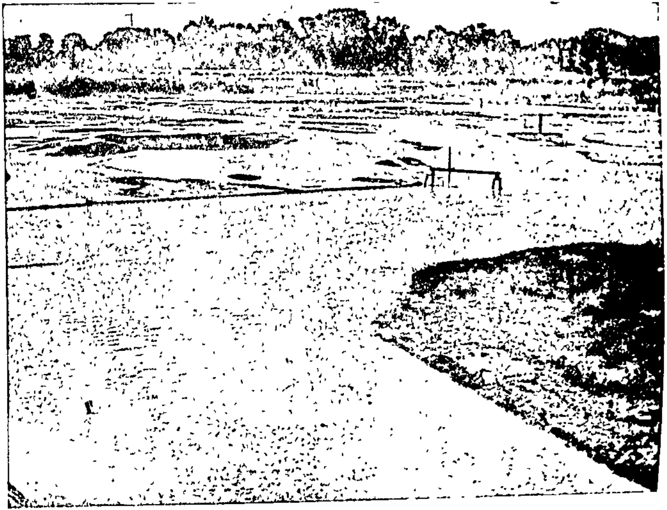
MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



Photograph showing the river in flood

FIG. 50

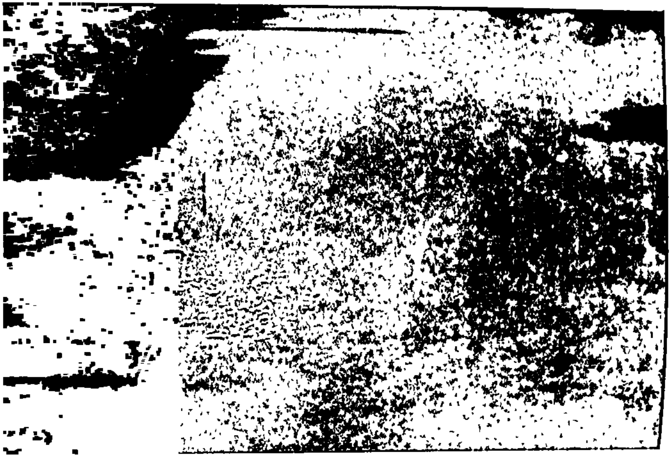
MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



The photograph illustrates the conditions of the river in the flood.

FIG. 51

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



The same with a higher discharge

FIG. 52

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

The photograph showing the condition at the line D. D. below discharges of 10,000 cusecs

FIG. 53

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

Photograph showing the condition at the line D. D. when both the channels flow in low discharges

FIG. 54

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

Photograph showing the conditions at the line D. D. in high discharges when both the channels flow

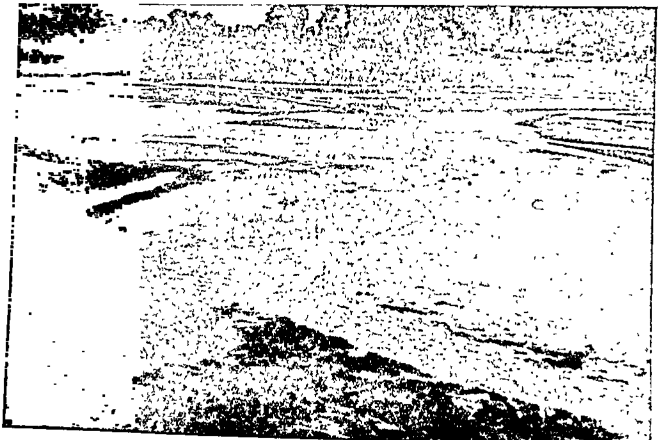
FIG. 55

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

The conditions of flow at the line D. D. just before giving way

Fig. 56

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



The photograph showing the conditions of flow after the junction took place

FIG. 57

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

The photograph illustrating the condition of flow at line D. D. after the junction of two channels in high discharges. It will be seen that major portion of the discharge still takes place in the central channel along a curved path.

FIG. 58

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

The conditions of flow in the right branch in higher discharges

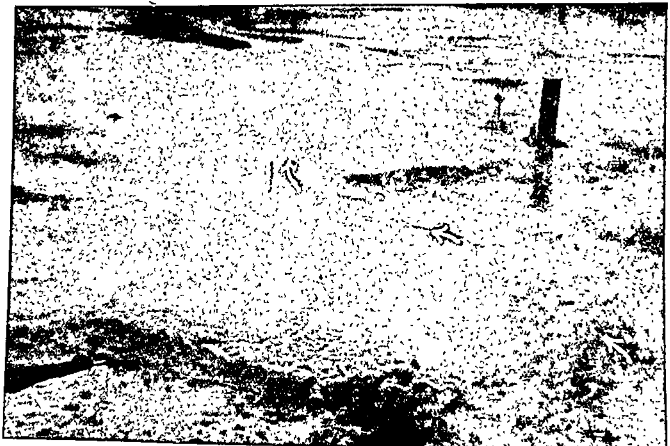
FIG. 59

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

The same as Fig. 57 but in higher discharges

Fig. 60

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



Photograph showing the direction of flow over the bela near the line D. D. on the right branch side

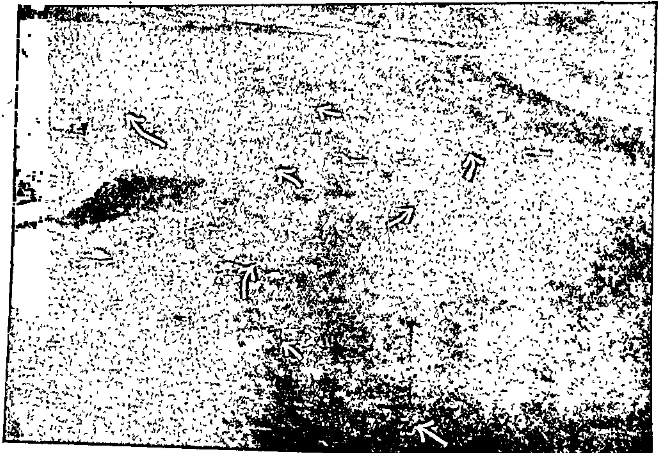
Fig. 61

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS

Showing the conditions of flow near the line D. D. in the right branch during the flood discharge of 200,000 cusecs

Fig. 62

MODEL OF RIVER SUTLEJ ABOVE SULEIMANKE HEADWORKS



Photograph showing course of flow after spilling over from the central channel to the right branch. It will be noted that the tendency of the water is to flow back towards the central channel.

creek for a short distance the water returned to the main channel. This is illustrated in Figs. 61 and 62.

It was decided to make a detailed study of this point. In order to do this, accurate levels of the *bela* between the two creeks were required. The Executive Engineer, Suleimanke Division, was requested to supply this information.

The Sub-Divisional Officer, Suleimanke Headworks, informed that the *bela*, where the distance between the central channel and the right creek was a minimum, was at an R. L. of 575·7. This information was considered to be insufficient as the Sub-Divisional Officer gave only one level of the *bela*. He was, therefore, again asked to send a detailed survey of that area. In the meanwhile the model was moulded according to the survey of November 1940 and the portion of the *bela* at the line DD′ was moulded to R. L. 575·7. A view of the moulded model of this portion is given in Fig. 52. The discharges examined before were run. The gauges observed at the nose of the earthen bund are given in Table VII.

The velocity observations in the central channel opposite the nose of the earthen bund were also made for discharges above 10,000 cusecs and are shown in Table VIII.

It will be seen from an examination of Tables VII and VIII that with very high discharges there is no considerable rise either in the gauge at the nose of the bund or increase in the velocities. This is due to the fact that with high discharges the water spreads all over the place.

Gauges were observed in the right creek and in the central channel opposite the line DD′. The water levels were also taken at R. D. 39,000 in the right creek and in the central channel. The observations on these points are given in Table IX.

It will be seen from Table IX that for medium discharges between 10,000 and 50,000 cusecs the difference in the water level between the central channel and the right creek is 1·5 feet to 1·7 feet approximately. With very high discharges the difference became negligible as the two channels joined. A detailed study of the conditions of flow at the line DD′ was made for different discharges. The conditions are illustrated in Figs. 52 to 61.

At the commencement of the experiment the difference between the right creek and the central channel was approximately 2·75 feet. With the increase in discharge in the central channel the right edge at the line DD′ started eroding as is shown in Figs. 53 and 54. The erosion of the bank continued on. With a discharge between 50,000 and 60,000 cusecs or in the neighbourhood of 60,000 cusecs the central channel just joined the right creek. This is illustrated in Figs. 54, 55 and 56.

FIG. 63

1940 BANK AND BED CONTOURS

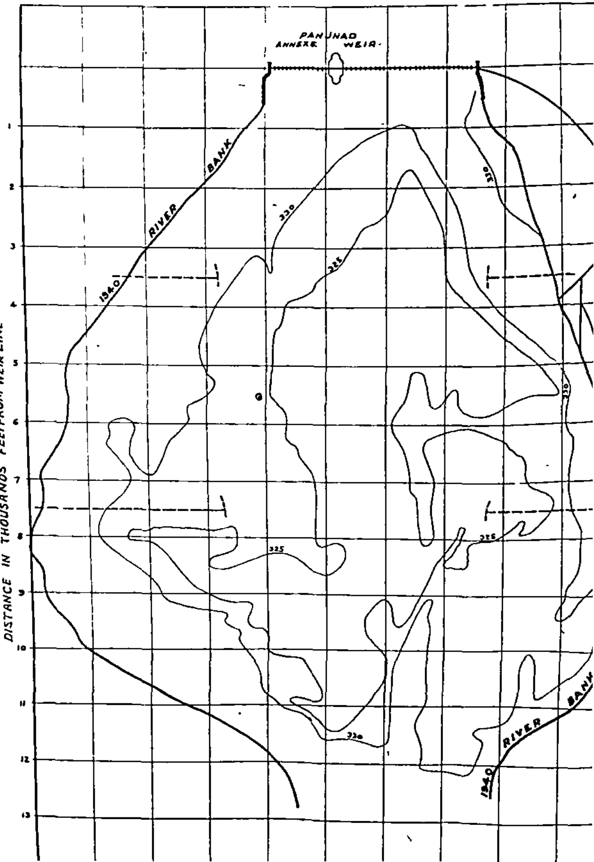


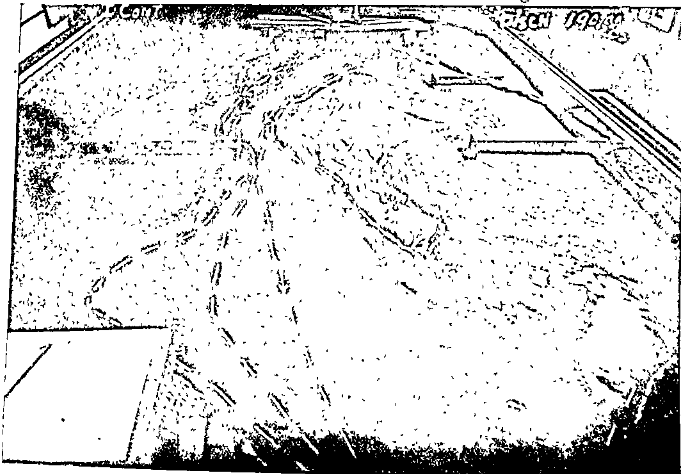
Fig. 64

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

Current directions with discharge equivalent to 8,000 cusecs

Fig. 65

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR



Current directions with discharge equivalent to 19,000 cusecs

FIG. 66

FOUR SPURS

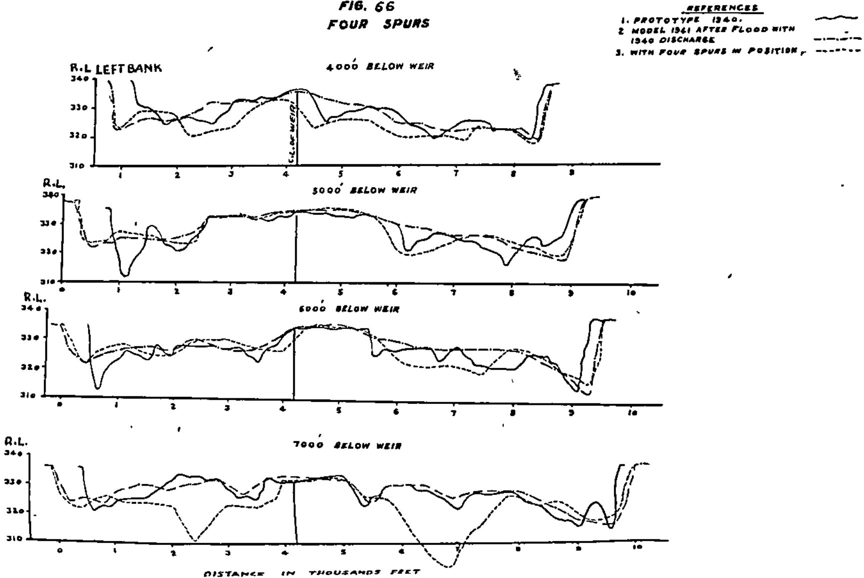
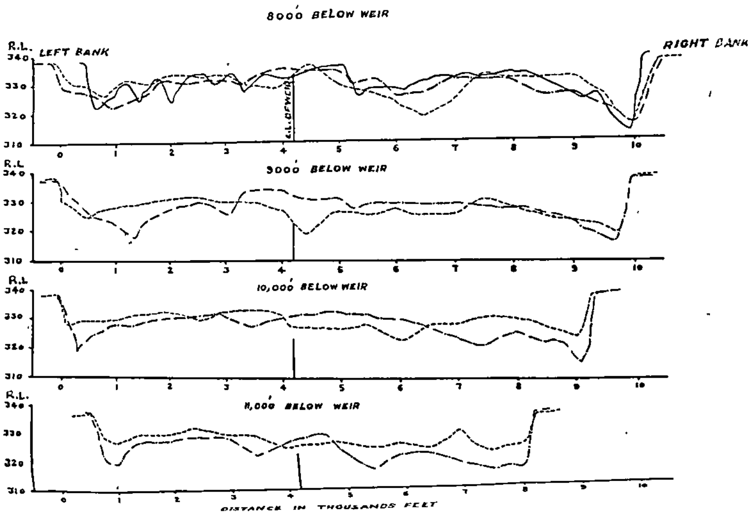
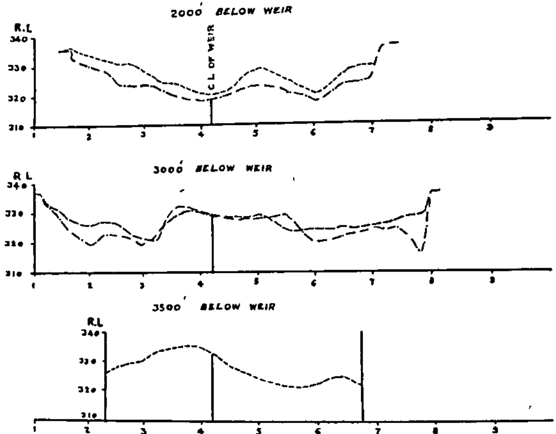


FIG. 66 (CONTD:)

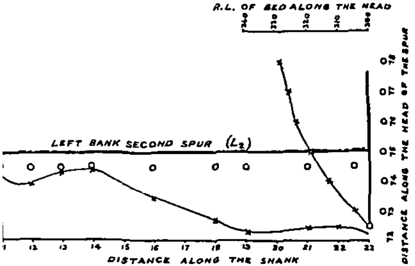


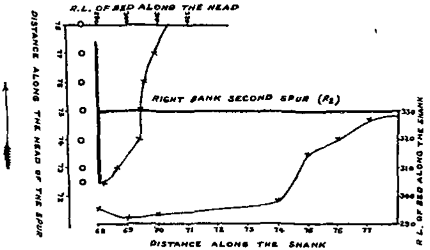
8000' BELOW WEIR
R.L.
LEFT BANK
RIGHT BANK
C.L. OF WEIR
9000' BELOW WEIR
10,000' BELOW WEIR
11,000' BELOW WEIR
DISTANCE IN THOUSANDS FEET

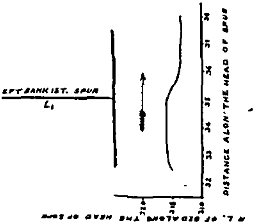
FIG.66 (CONTD:)



7500' BELOW WEIR (DISTANCE IN THOUSANDS FEET)


R.L. OF BED ALONG THE HEAD
LEFT BANK SECOND SPUR (L_2)
DISTANCE ALONG THE SHANK
DISTANCE ALONG THE HEAD OF THE SPUR


R.L. OF BED ALONG THE HEAD
RIGHT BANK SECOND SPUR (R_2)
DISTANCE ALONG THE SHANK
DISTANCE ALONG THE HEAD OF THE SPUR
R.L. OF BED ALONG THE SHANK


LEFT BANK IST. SPUR
L_1
DISTANCE ALONG THE HEAD OF SPUR
R.L. OF BED ALONG THE HEAD OF SPUR

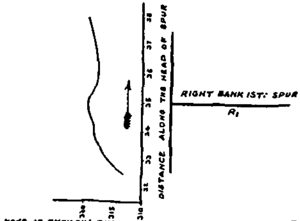

RIGHT BANK IST: SPUR
R_1
DISTANCE ALONG THE HEAD OF SPUR
R.L. OF BED ALONG THE HEAD OF SPUR

FIG 68

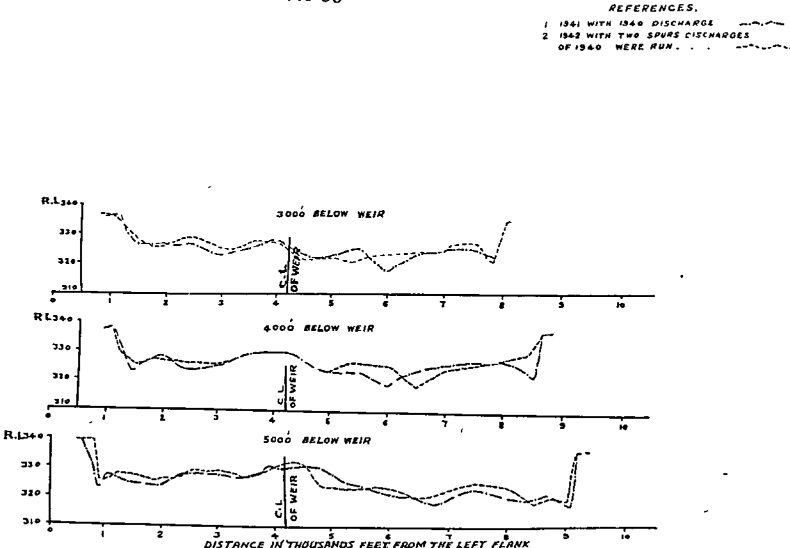
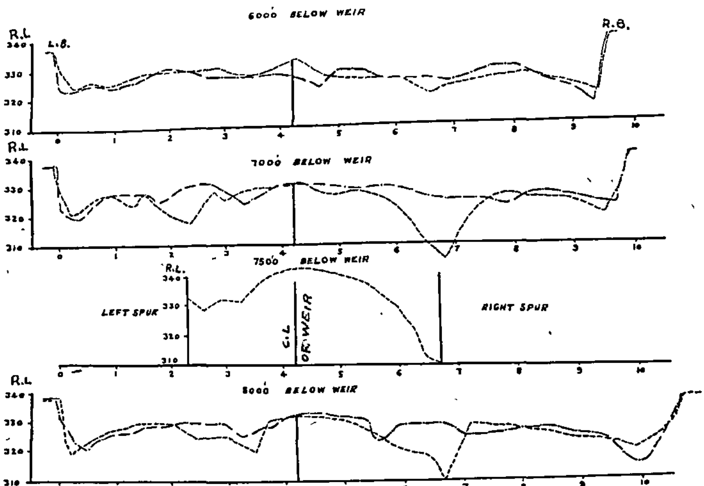
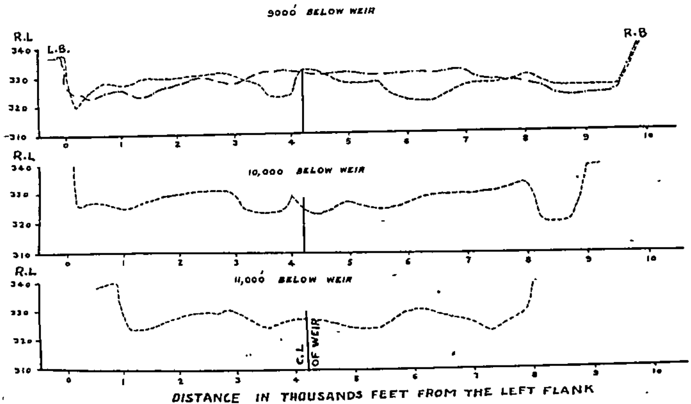

REFERENCES.
1 1941 WITH 1940 DISCHARGE
2 1942 WITH TWO SPURS DISCHARGES OF 1940 WERE RUN. . .
R.L 340
330
320
310
3000' BELOW WEIR
4000' BELOW WEIR
5000' BELOW WEIR
C.L OF WEIR
0
1
2
3
4
5
6
7
8
9
10
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

FIG. 68 (CONTD.)



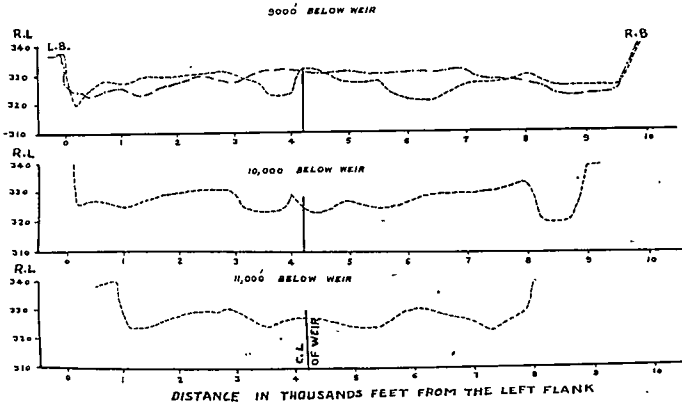
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

FIG. 68 (CONTD:)



9000' BELOW WEIR
10,000 BELOW WEIR
11,000' BELOW WEIR
R.L
L.B.
R.B
340
330
320
310
0
1
2
3
4
5
6
7
8
9
10
C.L OF WEIR
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

FIG.68 (CONTD:)



9000' BELOW WEIR
R.L
340
330
320
310
L.B.
R.B
0
1
2
3
4
5
6
7
8
9
10
10,000 BELOW WEIR
11,000' BELOW WEIR
C.L OF WEIR
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

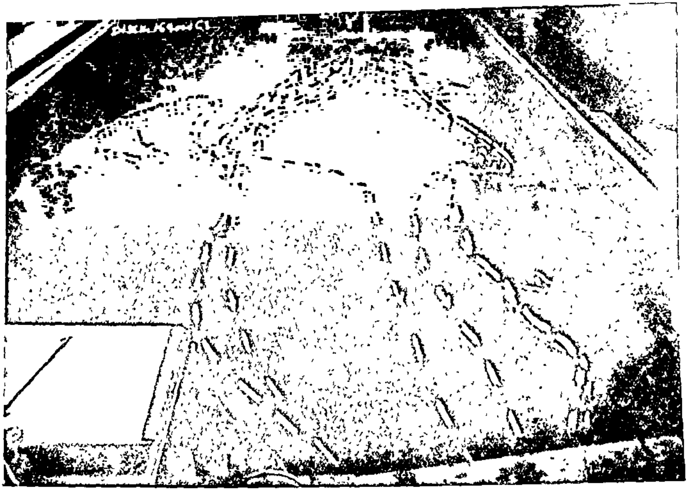
Fig. 69

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

Current directions with discharge equivalent o 100,000 cusecs

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

FIG. 70



Current directions with discharge equivalent to 164,000

FIG. 71 (CONTD.)

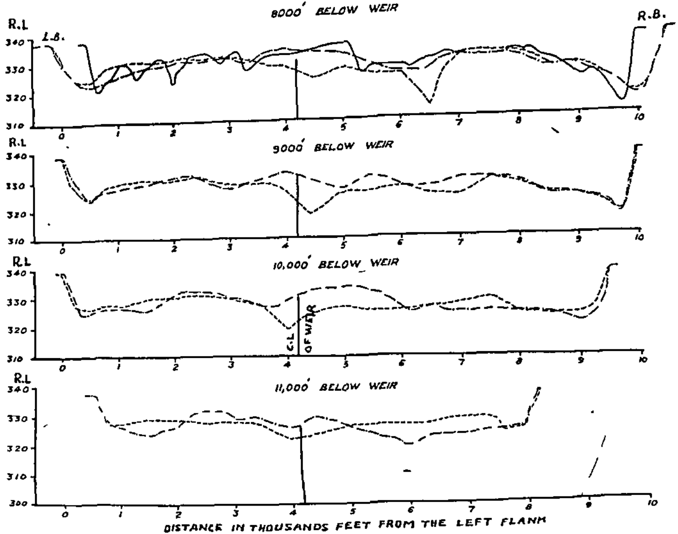
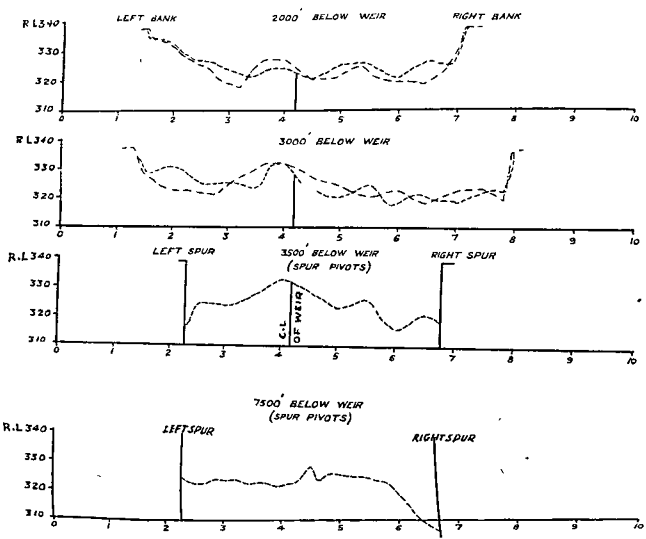

8000' BELOW WEIR
R.L
L.B.
R.B.
340
330
320
310
0
1
2
3
4
5
6
7
8
9
10
9000' BELOW WEIR
10,000' BELOW WEIR
C.L
OF WEIR
11,000' BELOW WEIR
300
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

FIG.71 (CONTD)



LEFT BANK
2000' BELOW WEIR
RIGHT BANK
R L 340
330
320
310
0
1
2
3
4
5
6
7
8
9
10
3000' BELOW WEIR
LEFT SPUR
3500' BELOW WEIR
(SPUR PIVOTS)
RIGHT SPUR
C.L
OF WEIR
7500' BELOW WEIR
(SPUR PIVOTS)
LEFTSPUR
RIGHTSPUR

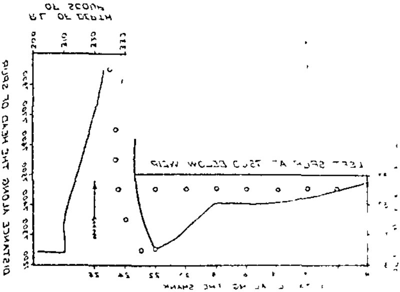
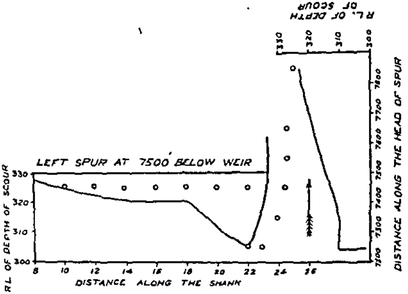
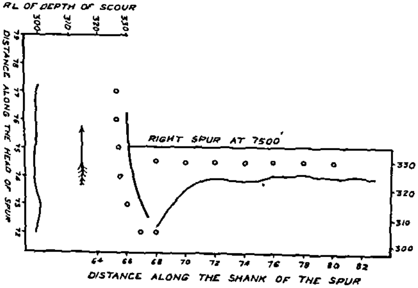

RL OF DEPTH
OF SCOUR
DISTANCE ALONG THE HEAD OF SPUR

FIG.72





EXPT. WITH TWO SPURS

FIG. 73
TWO SPURS

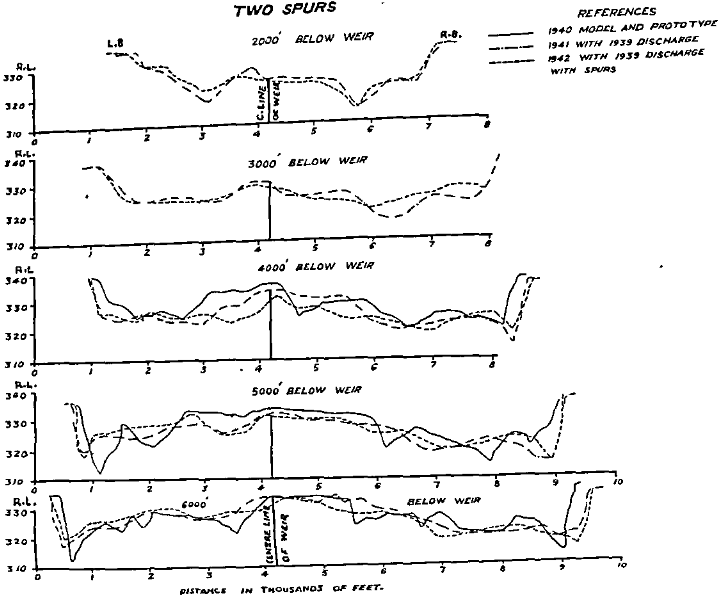


FIG.73 (CONTD)

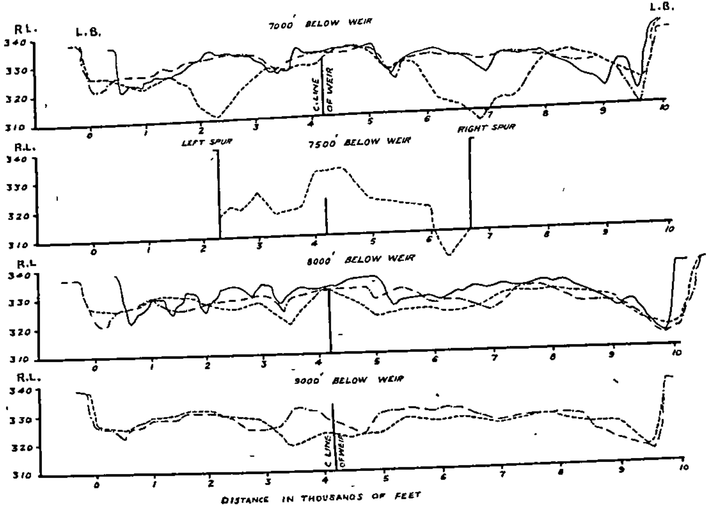
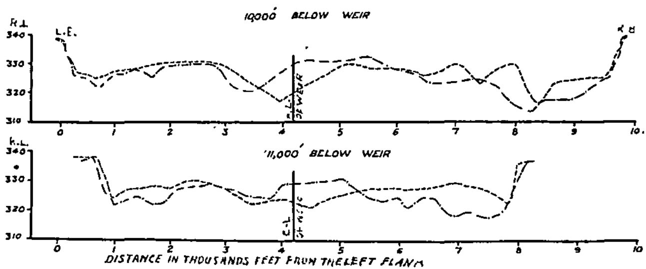

7000' BELOW WEIR
L.B.
R.L.
340
330
320
310
C.LINE OF WEIR
0 1 2 3 4 5 6 7 8 9 10
LEFT SPUR
7500' BELOW WEIR
RIGHT SPUR
8000' BELOW WEIR
9000' BELOW WEIR
DISTANCE IN THOUSANDS OF FEET

FIG.13 (CONTD)


10,000' BELOW WEIR
R.L.
L.E.
R.B.
C.L. OF WEIR
11,000' BELOW WEIR
C.L. OF WEIR
340
330
320
310
0
1
2
3
4
5
6
7
8
9
10
DISTANCE IN THOUSANDS FEET FROM THE LEFT FLANK

the year 1941 was assumed to be a year of low rivers while in Experiment No. 2 the year 1941 was assumed to be a year of high rivers. The results obtained in each case are discussed below :—

Experiment No. 1—Probable river conditions of the year 1941 *assuming low rivers*—The model was moulded to the survey of 1940 and discharges experienced in 1940 were selected to represent the 1941 discharges. Four spurs were constructed as shown in Fig. 63. The directions of flow at different stages were taken and are illustrated in Figs. 64 and 65. On the completion of the run, a bed survey was made and is plotted in Fig. 66. Scour along the heads of the spurs is also plotted in Fig. 67. The heading up caused by the spur with different discharges is given in Tables X and XI. The following indications were obtained :—

(1) With the four spurs in position there was no further bank erosion. The action was very light on the first pair of spurs.

(2) The downstream pair of spurs was heavily attacked in the beginning of the run. There was some flow along the shank of the second spur on the left bank. This flow obtained a kick from the head of the spurs towards the *bela*.

(3) The banks of the river were protected up to 7,000 feet from the weir.

(4) A deep channel formed along the line of the heads of the spurs. The central *bela* became reduced in size.

In the further tests the first two spurs were removed and experiments were again carried out as before. The results obtained with this arrangement are shown in Fig. 68 and are given below :—

(*i*) No erosion took place on either side up to 3,500′ from the weir.

(*ii*) Downstream of this point there was erosion of both banks.

(*iii*) The central *bela* extended in width and the confluence of the two channels moved downstream.

It was concluded that the most satisfactory results were obtained with four spurs.

Experiment No. 2—In the second experiment the bed was moulded to the survey of November 1940 and discharges experienced in 1939 were run to represent a high river year. The tests were carried out as before. The current directions obtained with these discharges are illustrated in Figs. 69 and 70. The survey obtained at various points and scour along the heads of the spurs are plotted in Figs. 71 and 72. As before, tests were carried out with two and four spurs. The results obtained with two spurs are plotted in Fig. 73. In the presence of the four spurs the following indications were obtained :—

(1) An attack occurred on the left bank below 4,000 feet from weir.

(2) Concentration of flow took place at 10,000 feet along the left bank from weir.

(3) The confluence of the two channels moved downstream as observed in the first experiment.

When the two spurs were in position the following points were noticed :—

(i) Upstream of the spurs there was not much erosion though the flow occurred along the bank.

(ii) Heavy action took place at the nose of the spurs.

As a result of the experiments carried out it was recommended that four spurs should be constructed at positions shown in Fig. 74.

The Chief Engineer inspected the model of the Panjnad Headworks during his monsoon tour on the 16th of July 1941 and discussed the results obtained on the model. During the course of the discussion it was mentioned to the Chief Engineer that—

(1) the points of maximum erosion as now obtained moved farther down than those obtained in the previous years ;

(2) the junction of the two channels shifted downstream ; and

(3) the position of the central *bela* downstream of the weir for a length of 5,000 feet was not much altered.

It was pointed out to the Chief Engineer that if the erosion downstream of the weir farther than 5,000 feet did not matter much the construction of the spurs may be postponed. However, the probable river conditions at the end of years 1941, 1942 and 1943 were determined on the model. A comparison of the river conditions from year to year for six years was made.

In Fig. 75 the river edge from 1937 to 1940 as obtained on the prototype is plotted. The central *bela* for 1937 and 1938 is also plotted on this figure. It will be seen from an examination of this figure that there is a considerable erosion in the year 1939 downstream of the weir at both the banks. In the years 1939 and 1940 there has been practically no change in the position of the eroded edge of the left bank. This may be due to the construction of temporary spurs along the left bank but on the right bank also up to a length of 4,000 to 5,000 feet from the weir there is little difference from the previous year. The maximum shifting of the bank, however, occurred at 8,500 feet from the weir.

In Fig. 76 the central *bela* for 1938 and 1939 is plotted. It will be seen that the *bela* extended downstream from 10,000 to approximately 12,000 feet but the conditions of the *bela* with regard to the central line of the weir were not altered.

In Fig. 77 the conditions of the *bela* in 1939 and 1940 are given. This figure shows that there is no considerable change in the position of the *bela* in 1939 and 1940. The *bela* did not extend in 1940 much further downstream than where it was in 1939.

FIG. 74

SUTLEJ RIVER BELOW PANJNAD WEIR

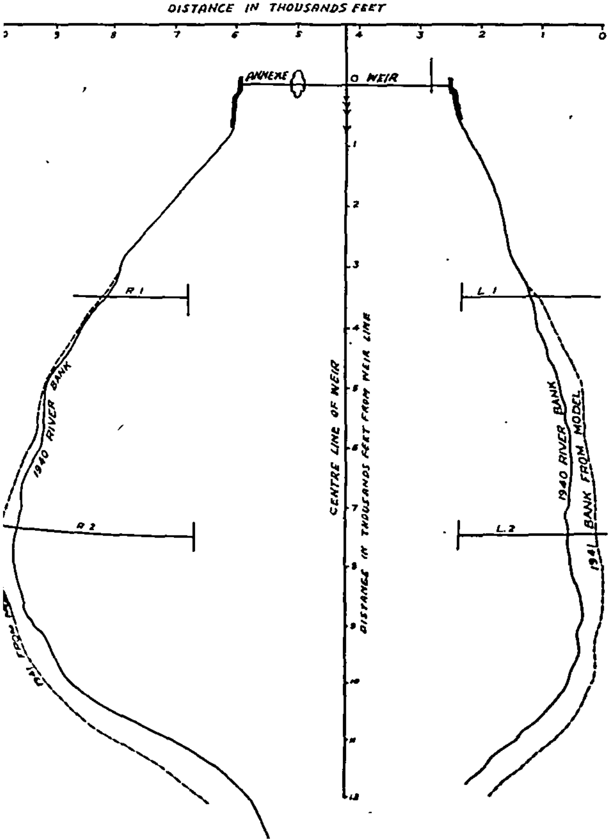


FIG. 15

CONTOUR SURVEY OF PANJNAD RIVER

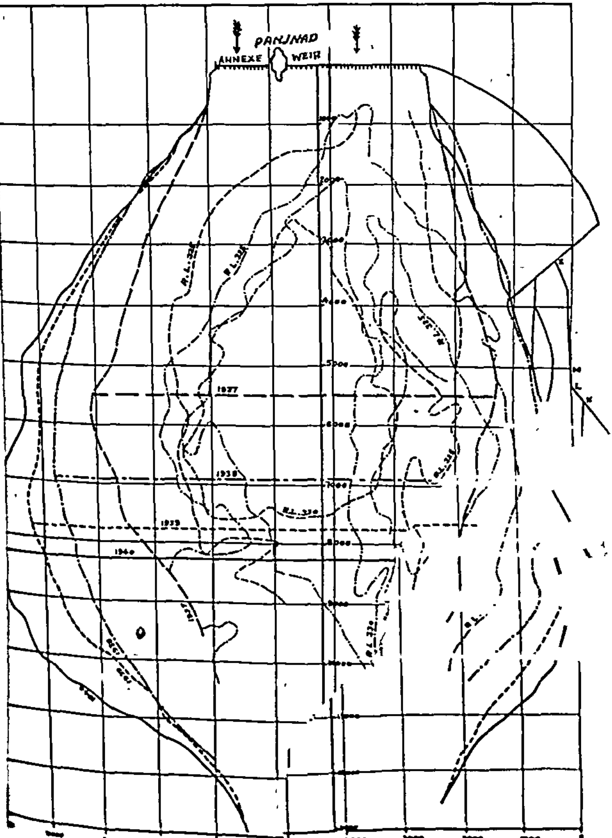
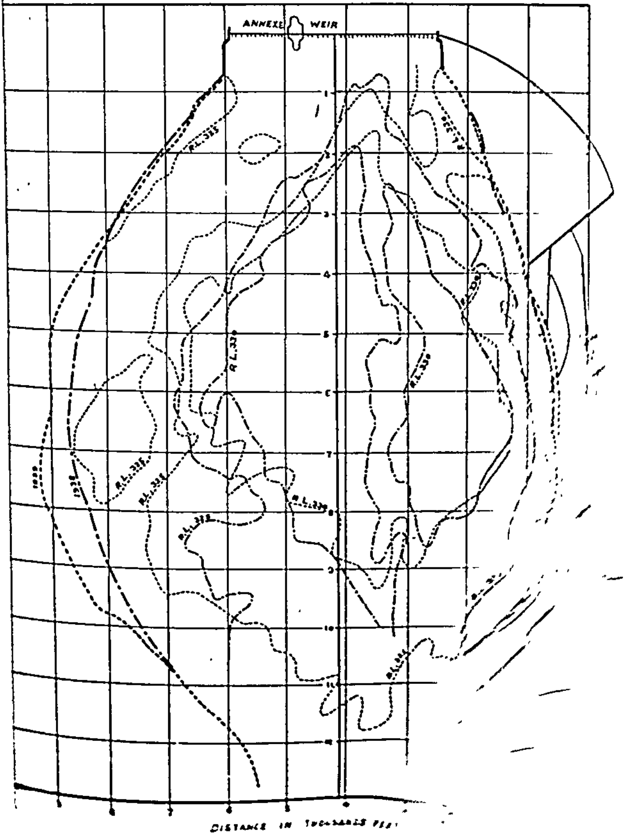
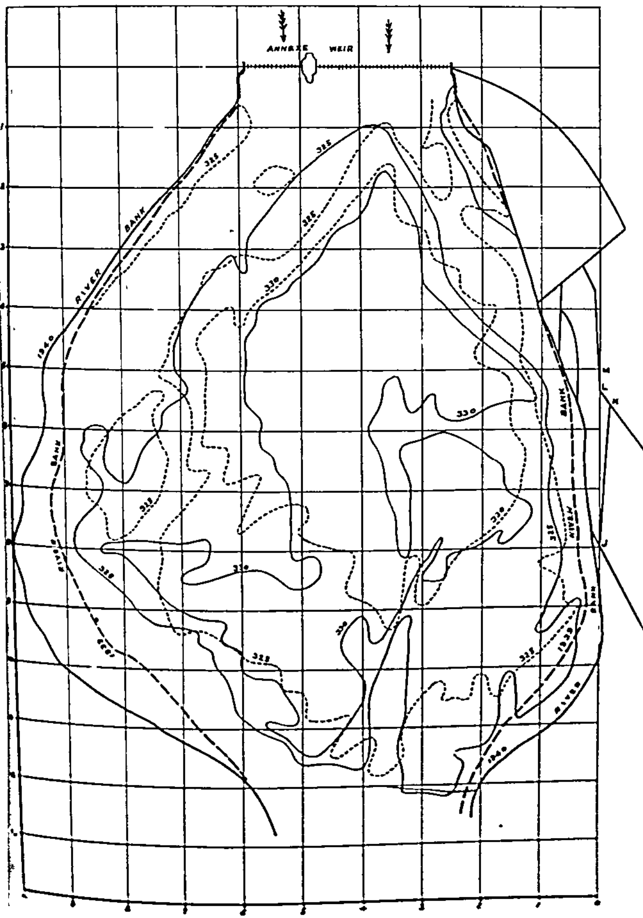


FIG. 76

CONTOUR SURVEY OF PANJNAD RIVER





DISTANCE IN THOUSANDS FEET

FIG. 77

CONTOUR SURVEY OF PANJNAD RIVER

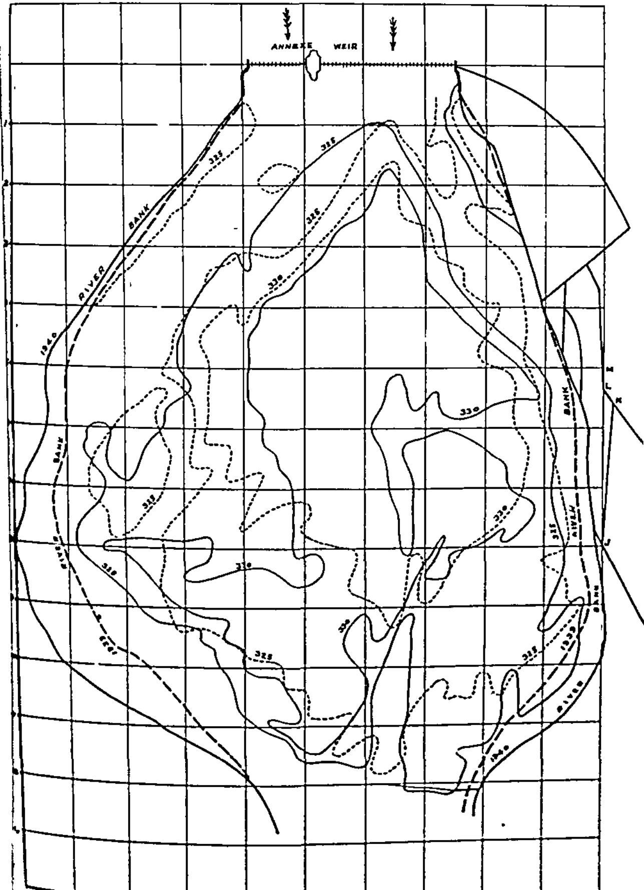


FIG.78

BANK EROSION BELOW PANJNAD WEIR

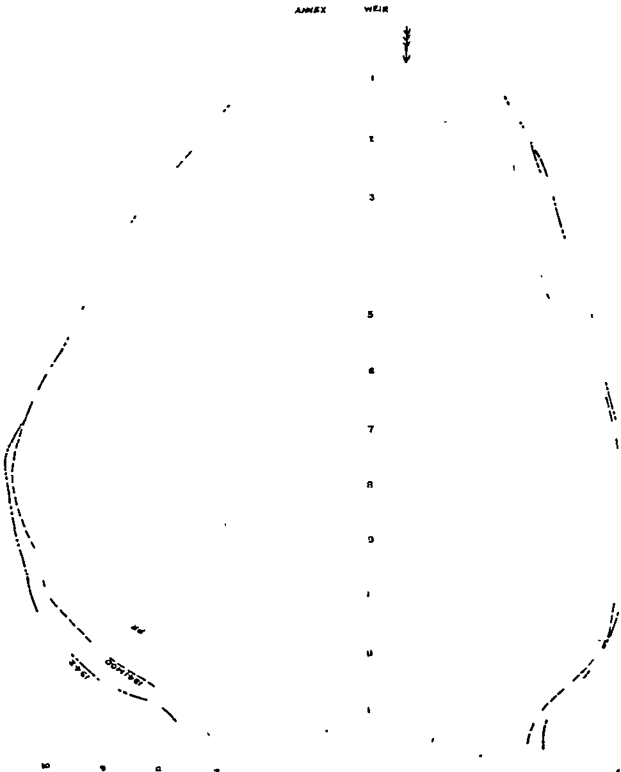
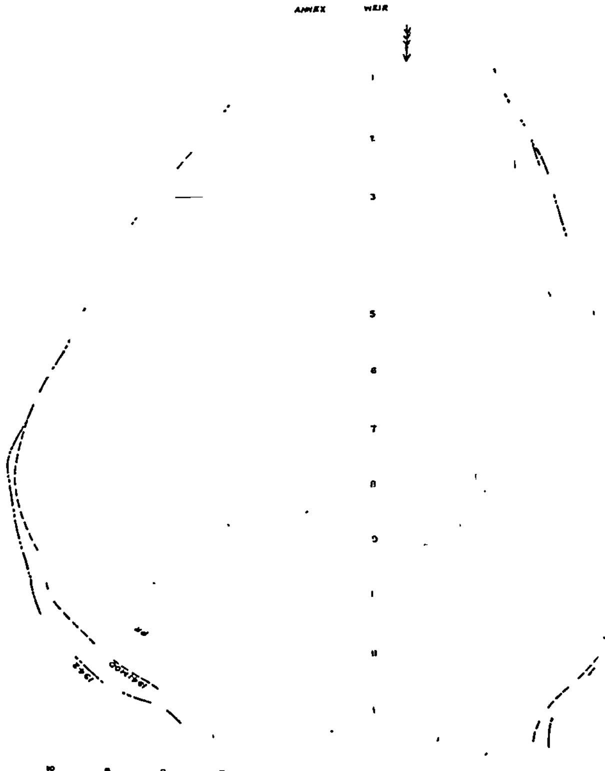
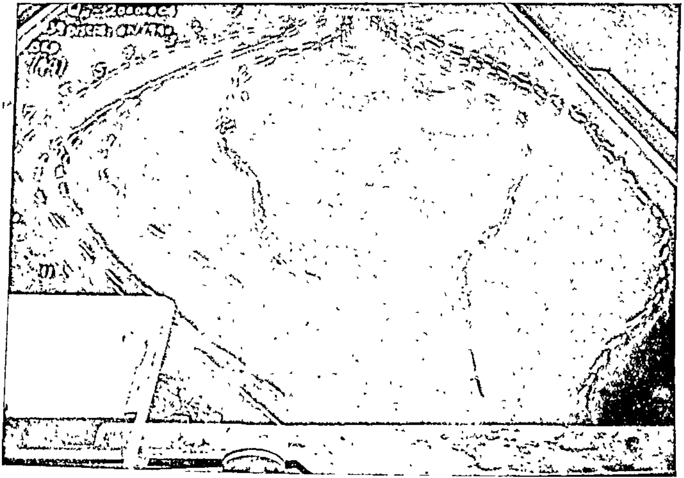


FIG.78

BANK EROSION BELOW PANJNAD WEIR



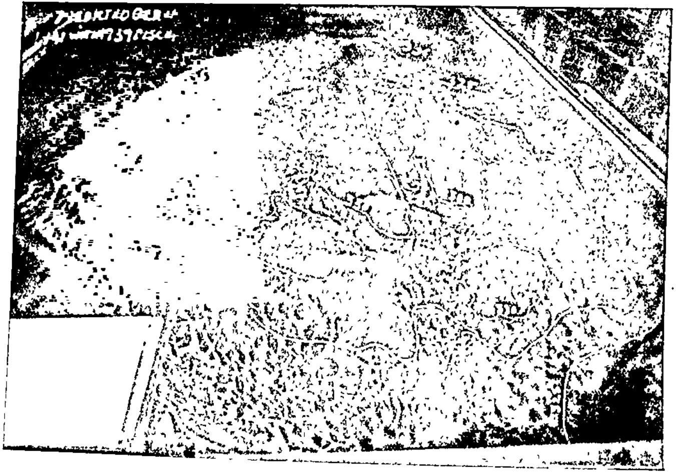
MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR



Forecast of the direction of flow at the end 1941

FIG. 80

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR



Probable condition of river bed at the end of 1941

FIG. 81

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

Forecast of the direction of flow at the end of 1942

FIG. 82

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

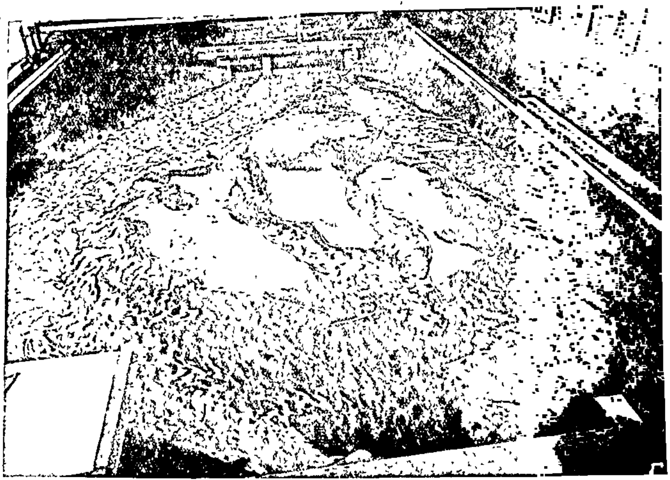
Probable condition of river bed at the end of 1942

FIG. 83

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR

Fig. 84

MODEL OF RIVER PANJNAD BELOW PANJNAD WEIR



Probable condition of river bed at the end of 1943

FIG. 86

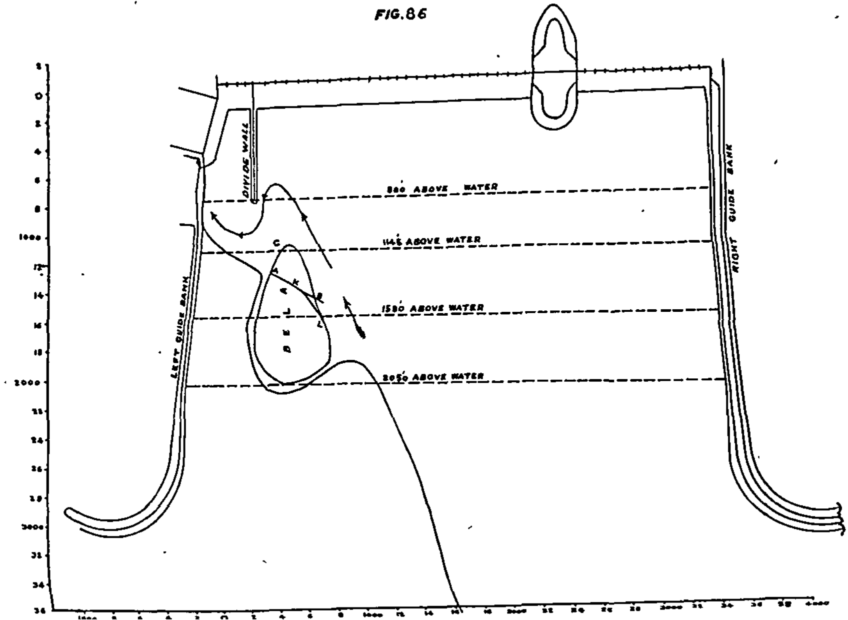
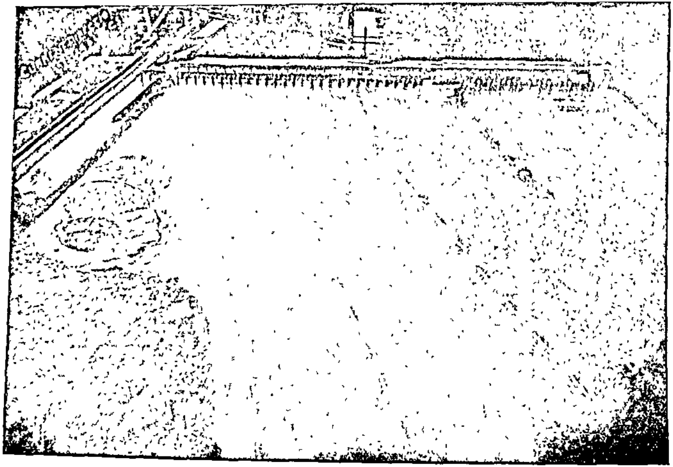

DIVIDE WALL
880 ABOVE WATER
1145 ABOVE WATER
1580 ABOVE WATER
2050 ABOVE WATER
LEFT GUIDE BANK
RIGHT GUIDE BANK
C
A K E
L
L
D E L

FIG. 87

MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS



Showing the directions of flow with a discharge of 150,000 cusecs under existing conditions.

The probable river edge in the years 1941, 1942 and 1943 was determined from the model and the position of the left and the right banks for different years is plotted in Fig. 78. From an examination of this figure it will be seen that for the years 1941 and 1942 the river edge on both sides up to 7,000 feet is not much altered. After 7,000 feet there is some erosion of the bank but that also is not much, and takes place downstream of 8,000 feet from the weir line. The current direction and the bed conditions at the end of 1941, 1942 and 1943 are also illustrated photographically in Figs. 79 to 84.

It may be pointed out that for protection of the banks up to 5,000 to 6,000 feet downstream of the weir armoured spurs may not be much needed, as is indicated by the conditions of flow. Any temporary spurs constructed on the left bank should be maintained. However, if it is desired to protect the banks much farther downstream and to train the river into a single channel the spurs are necessary. The conditions at site regarding the bank erosion taking place and any development of the creeks in the central *bela* would further help in arriving at the decision.

IV—A Study of a model of Panjnad Headworks to determine the effect of scraping the bela *upstream of the left divide wall on the river approach to the left pocket.*

Rai Bahadur Lala Ganpat Rai, Officer on Special Duty, visited the Hydraulic Laboratory about the 7th of December 1941 and asked the proposals formed by the Superintending Engineer, Haveli Canal Circle of scraping the *bela* opposite the left divide wall in order to obtain an easy approach to the left pocket to be tested. The proposal is illustrated in Fig. 85. The portion of the *bela* to be scraped is marked in red on this figure.

In order to test the above proposal a large scale model of the Panjnad Headworks the canal regulator and one mile of the river

As no other data was available to further check the model in detail the tests were commenced.

Effect of scraping the bela—The *bela* shown A K L B C A on the plan in Fig. 85 was scraped by 5 feet and the model was run with varying discharges. The directions of flow were again observed with a discharge of 150,000 cusecs and are shown in Fig. 88. A comparison of Figs. 87 and 88 will show that no appreciable difference in the directions of flow is obtained when the *bela* is scraped off. A very feeble current exists on the scraped portion which lies on the inside of the curve. Due to these conditions there is a deposit of silt taking place on the surface of the *bela*.

After running the model for 10 hours, which is equivalent to one season, the model was stopped and an inspection of the bed was made. The conditions of the scraped *bela* are illustrated in Fig. 89. From an examination of this figure it will be seen that the scraped portion is again silted up.

In the next test the model was again moulded to the given sections and 10 feet was scraped off the *bela*. Discharges similar to those used before were run. Current directions were taken at different stages of the experiment and are shown in Figs. 90 to 94. It will be seen from an examination of Figs. 90 to 94 that with the *bela* scraped by ten feet, the concentration of flow at the nose of the left divide wall and swirl formation on the scraped portion of the bela still take place. From Figs. 94 and 95 it is clear that the scraped portion has again silted up.

It can, therefore, be concluded from the above tests that no useful purpose will be served by scraping 5 feet or 10 feet of the *bela* as proposed by the Superintending Engineer, Haveli Canals Circle.

The Chief Engineer, West, inspected the model at different stages of the investigation.

Further experiments to determine the effect of the regulation of the weir on the oblique approach of the river to the left pocket, are being carried out. As soon as the results are available a report will be submitted.

V—Training of the River Ravi above Madhopur Pocket.

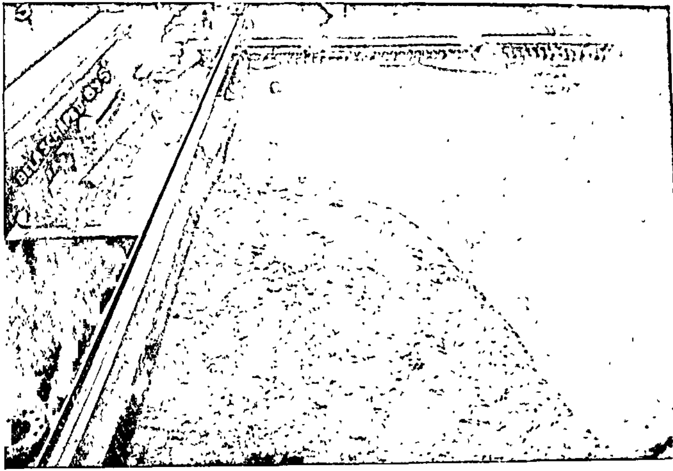
The river conditions obtained at the end of the monsoon of 1941 were far from satisfactory. A large *bela* had developed in front of the Marginal bund. In high discharges this *bela* remained under water, but for most of the period in winter the *bela* was above the water level. The extension of the *bela* resulted in choking the channel flowing along the Marginal bund.

The Chief Engineer asked for experiments to be made to determine methods to erode the *bela*. Accordingly the following tests were made on the Madhopur pocket model :—

I. Testing the effect of construction of a divide groyne.

Fig [illegible]

MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS



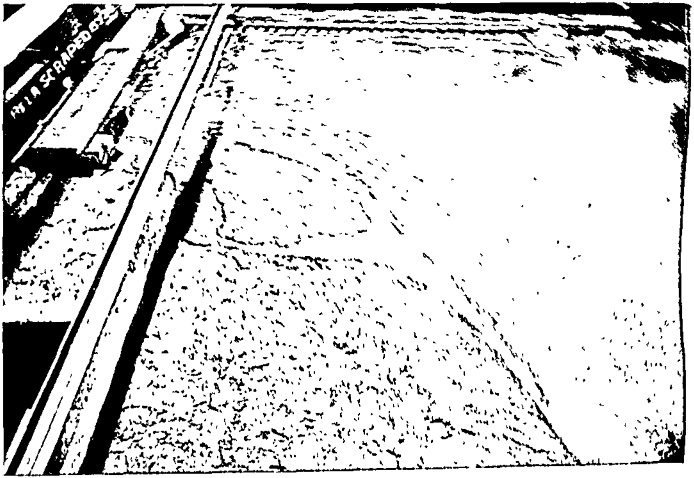
Showing current directions after having scrapped off the proposed portion of the bela opposite the left divide wall. It will be seen that directions of flow in this case are similar to those given in Fig. 87 when the bela is not scrapped

FIG. 85

MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS

Showing the bed after running the model. It is indicated from this photograph that the portion scrapped off tends to silt up.

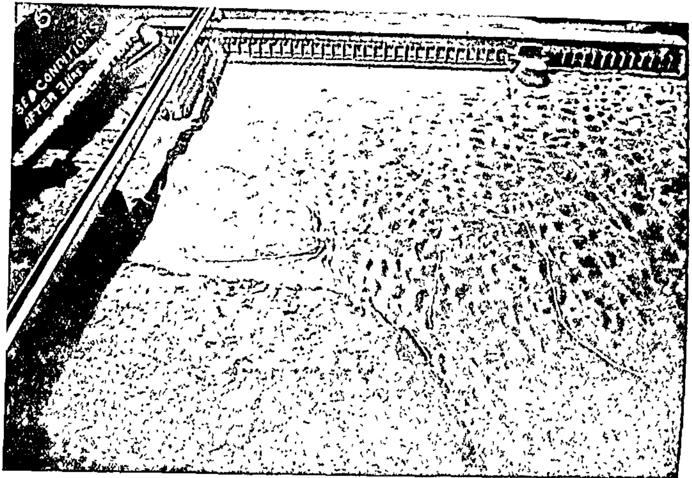
MODEL OF PANJNAD HEADWORKS AND [illegible]



Showing the current directions in the initial stages of experiment after scrapping the bela by 10′ instead of 5′ as in the previous cases. In the early stages there is some tendency of flow taking place over the scrapped portion of the bela.

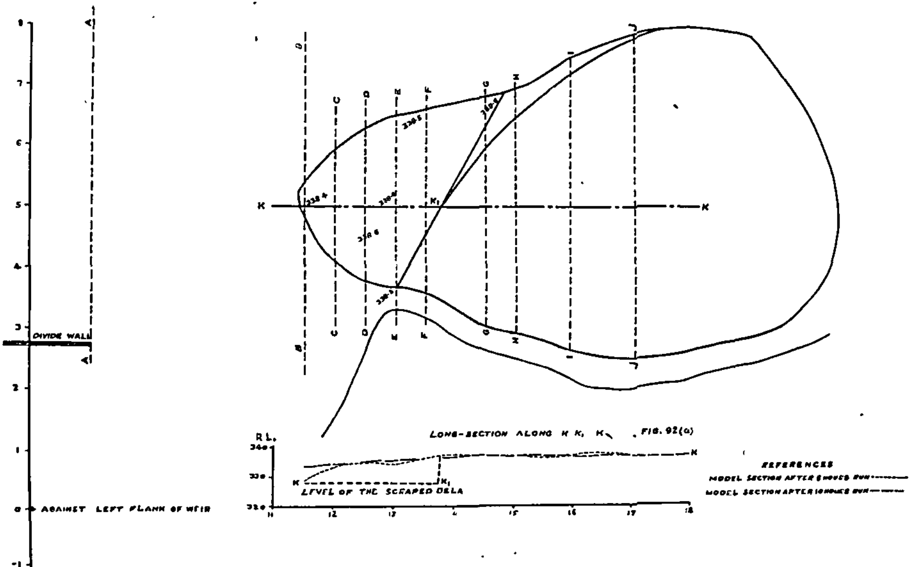
FIG. 91

MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS



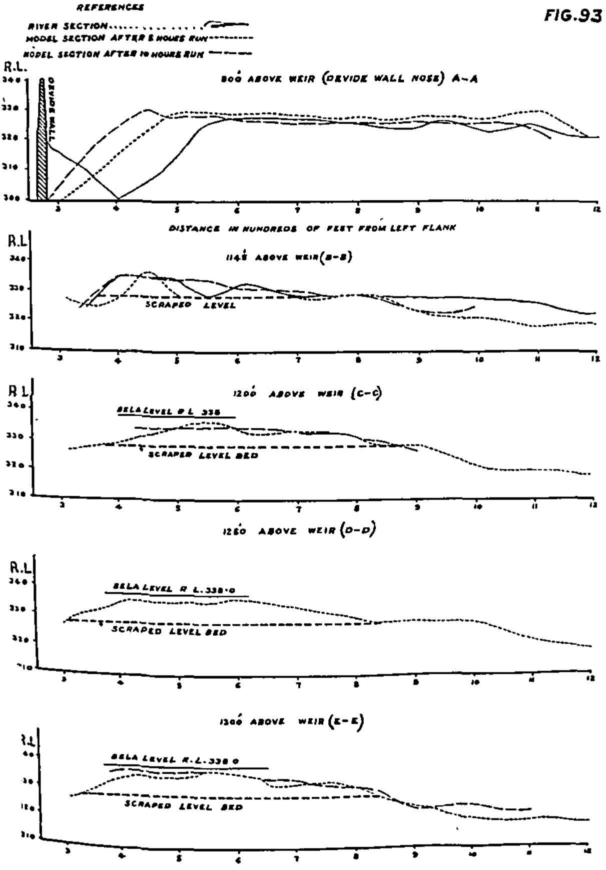
Showing the river bed after 3 hours' run. The silting of the bela commences

FIG.92



DIVIDE WALL
AGAINST LEFT FLANK OF WEIR
LONG-SECTION ALONG K K1 K , FIG. 92(a)
R.L.
LEVEL OF THE SCRAPED DELA
REFERENCES
MODEL SECTION AFTER 8 HOURS RUN
MODEL SECTION AFTER 10 HOURS RUN

FIG.93



REFERENCES
RIVER SECTION
MODEL SECTION AFTER 5 HOURS RUN
MODEL SECTION AFTER 10 HOURS RUN
R.L.
800 ABOVE WEIR (DEVIDE WALL NOSE) A-A
DEVIDE WALL
DISTANCE IN HUNDREDS OF FEET FROM LEFT FLANK
1145 ABOVE WEIR (B-B)
SCRAPED LEVEL
1200 ABOVE WEIR (C-C)
BELA LEVEL R L 338
SCRAPED LEVEL BED
1250 ABOVE WEIR (D-D)
BELA LEVEL R L. 338·0
SCRAPED LEVEL BED
1300 ABOVE WEIR (E-E)
BELA LEVEL R.L. 338 0
SCRAPED LEVEL BED

FIG. 93 (CONTD)

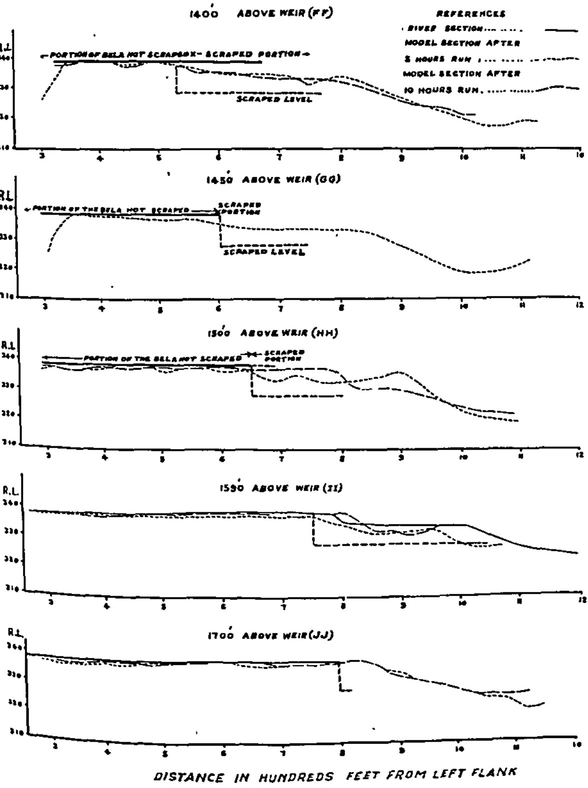
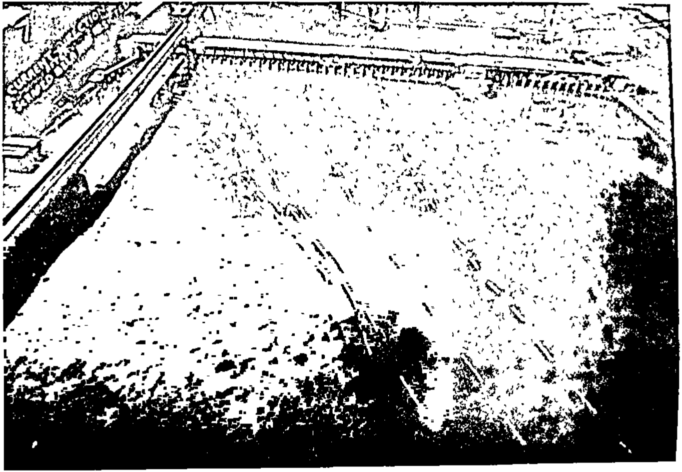

1400' ABOVE WEIR (FF)
REFERENCES
RIVER SECTION
MODEL SECTION AFTER 5 HOURS RUN
MODEL SECTION AFTER 10 HOURS RUN
PORTION OF BELA NOT SCRAPED
SCRAPED PORTION
SCRAPED LEVEL
R.L.
1450' ABOVE WEIR (GG)
PORTION OF THE BELA NOT SCRAPED
SCRAPED PORTION
SCRAPED LEVEL
1500' ABOVE WEIR (HH)
PORTION OF THE BELA NOT SCRAPED
SCRAPED PORTION
1590' ABOVE WEIR (II)
1700' ABOVE WEIR (JJ)
DISTANCE IN HUNDREDS FEET FROM LEFT FLANK

FIG. 91

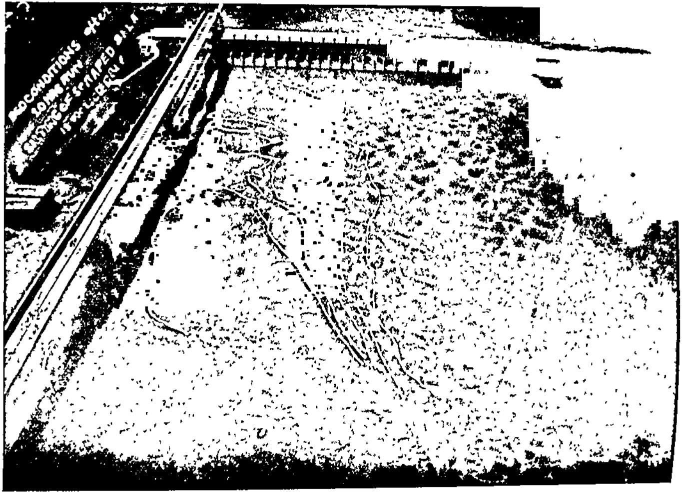
MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS



After 10 hours' run. The original tendency of flow to take place over the scrapped portion of the bela is considerably decreased

Fig. 95

MODEL OF PANJNAD HEADWORKS AND RIVER SUTLEJ ABOVE THE WORKS



Showing the silting up of the bela after 10 hours' run of the model

FIG. 96

MADHOPUR POCKET MODEL

X-SECTION SHOWING THE EFFECT OF DIVIDE WALL 350' LONG AT PIER NO 4

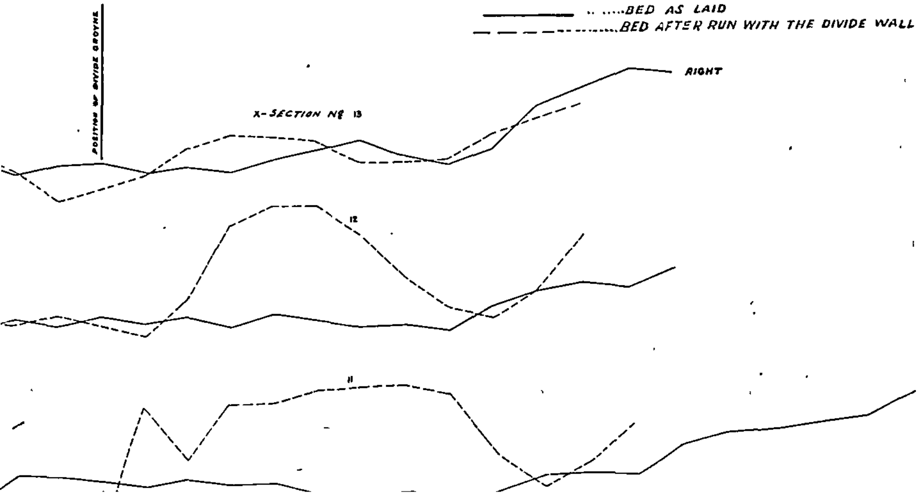


FIG. 96 CONTINUED

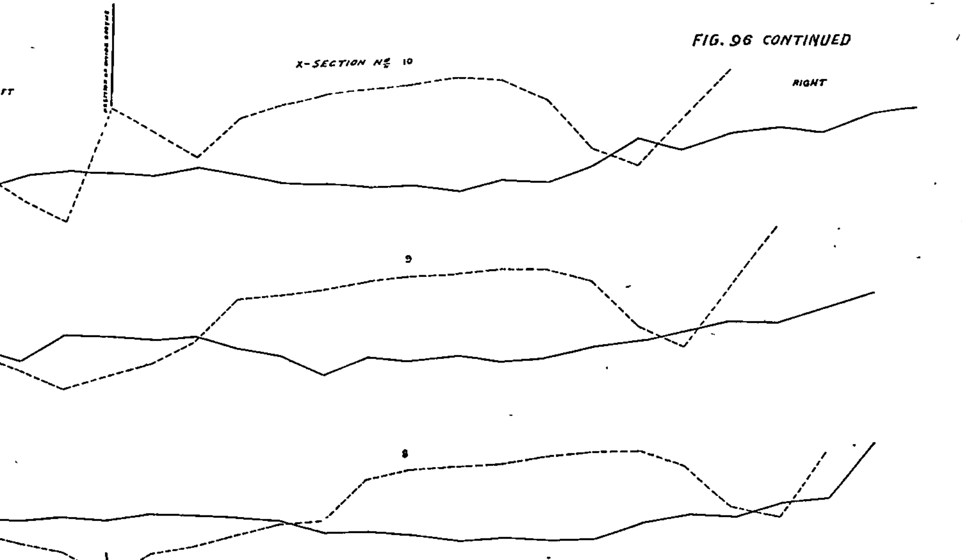
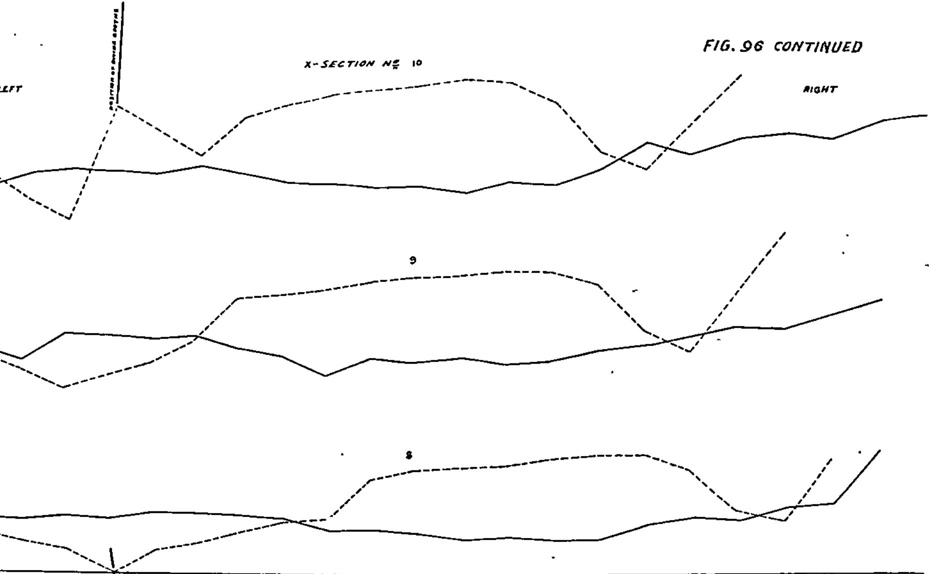
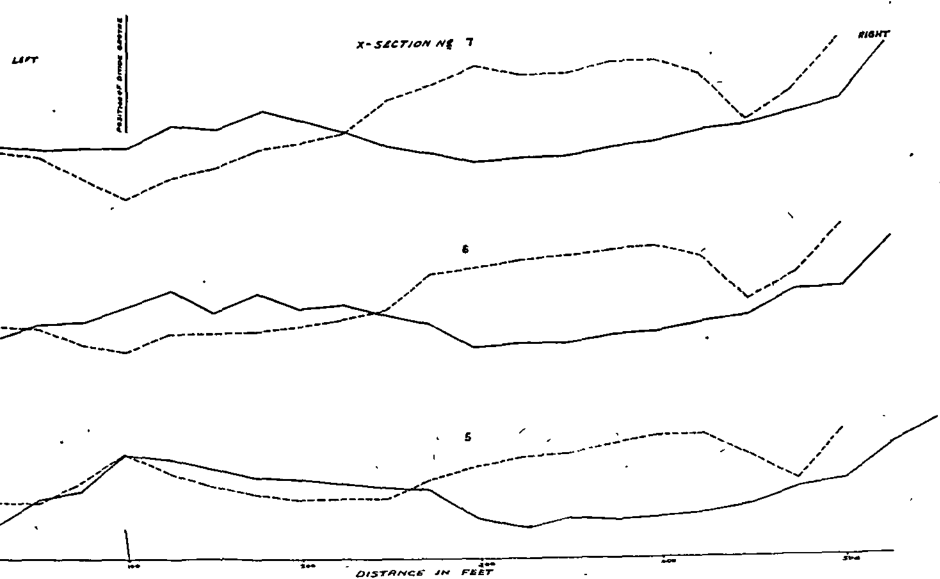
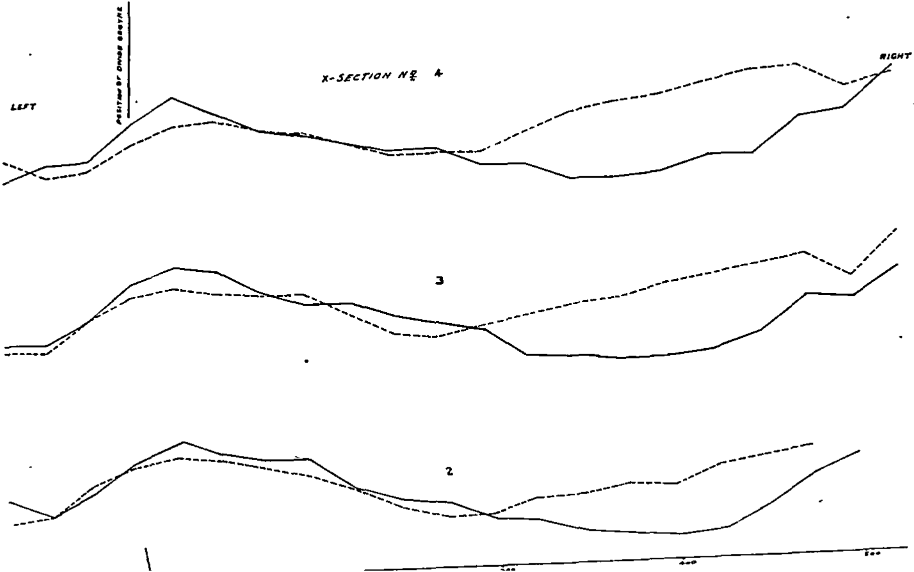


FIG. 96 CONTINUED





LEFT
X-SECTION No 7
RIGHT
6
5
DISTANCE IN FEET


X-SECTION Nº 4
LEFT
RIGHT
3
2

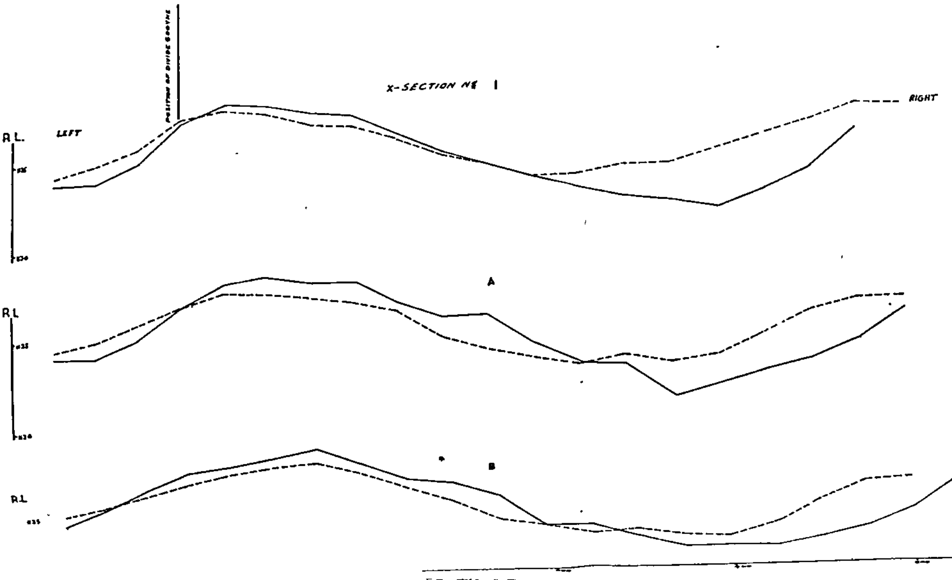

X-SECTION Nº 1
LEFT
RIGHT
R.L.
A
R.L.
B
R.L.

FIG. 97 CONTINUED

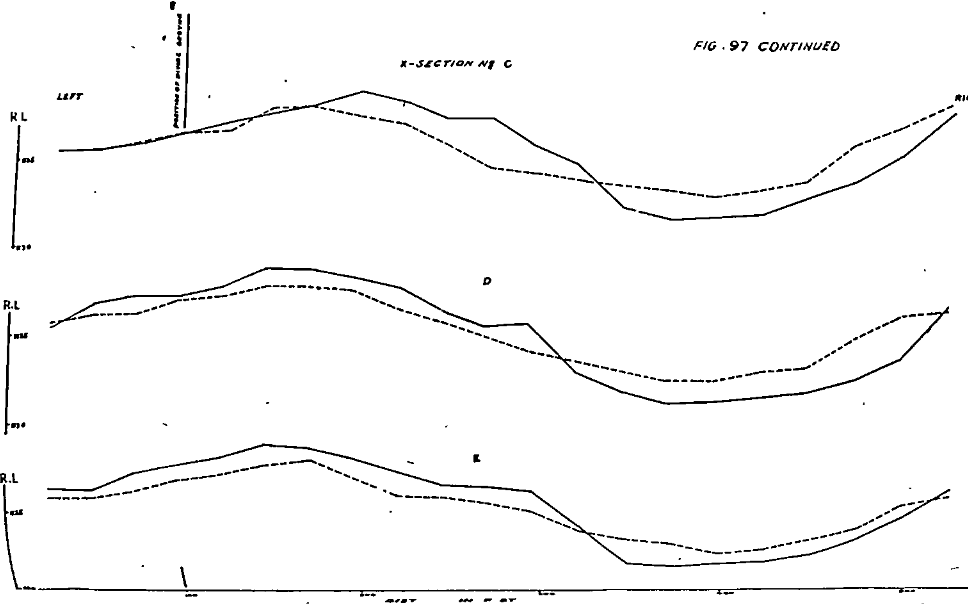


FIG. 97 CONTINUED

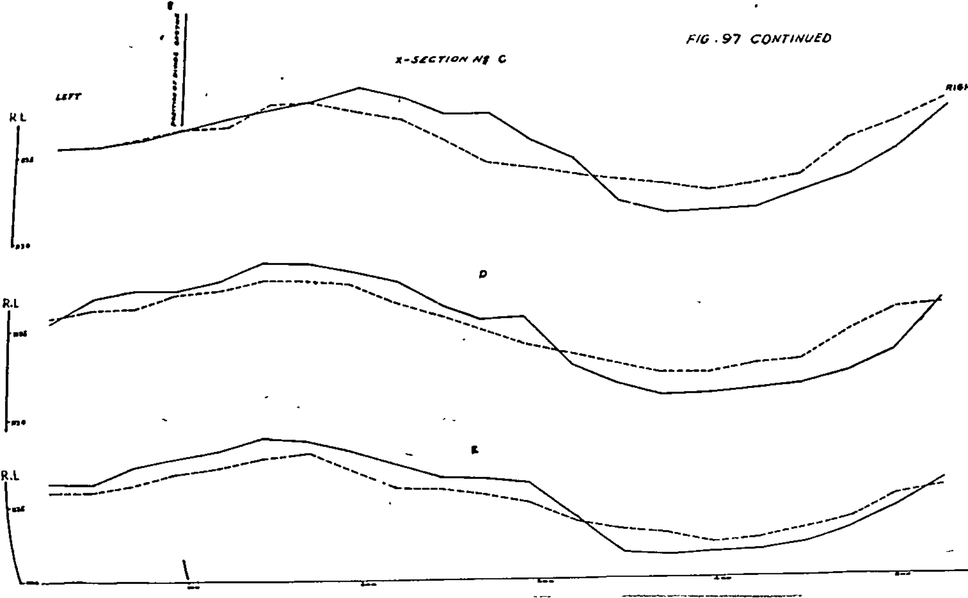


FIG 96 CONTINUED

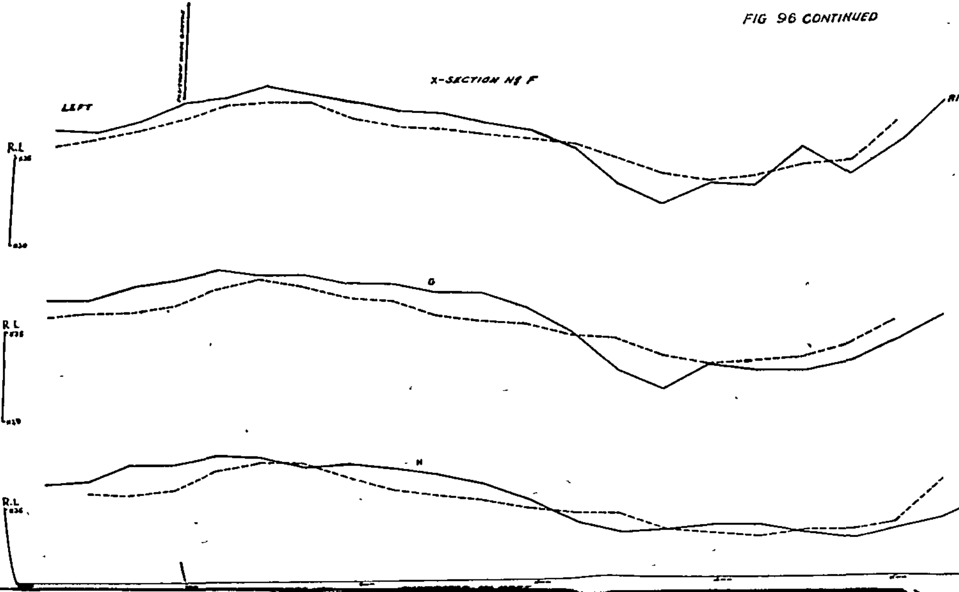


FIG 96 CONTINUED

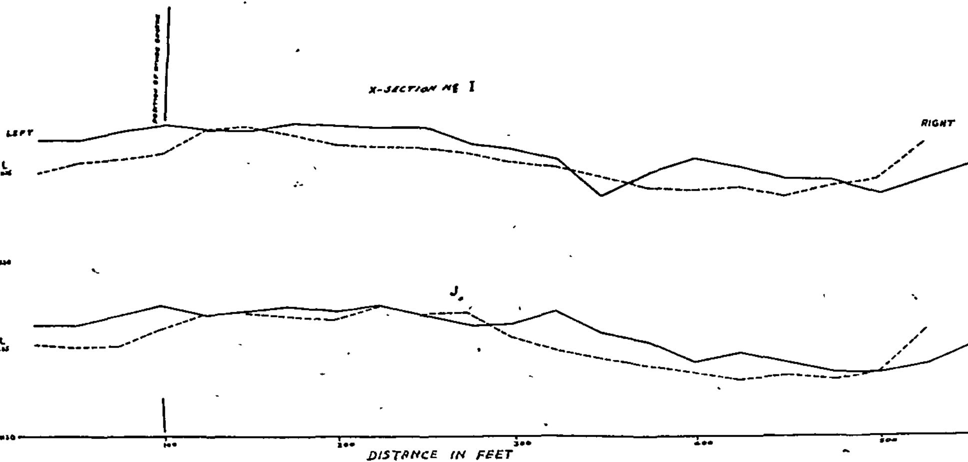


FIG. 97

MADHOPUR POCKET MODEL

SHOWING THE EFFECT OF CONSTRUCTING A DIVIDE WALL AT PIER NO 4

SILT IN C. F. ENTRING THE BAYS OF CANAL

DAYS	1	2	3	4	5	6	7	8	9	10	11	12		
SILT	.01	.03	.15	.34	.36	.92	.23	.29	.29	.39	.35	35 = 3.51	WITH DIVIDE WALL	ORIGINAL BED LAID AS IN 1940 / DIVIDE WALL FITTED / GATES - 1-4/NIL, WEDGE FROM LEFT
"	.02	.28	.60	.34	.76	.65	.31	.57	.56	.60	.51	51 = 6.14	" " "	GATES 1-4/NIL " " RIGHT
"	.06	.01	.01	.22	.17	.54	.70	.76	.68	.71	.69	85 = 4.24	NO " "	" CENTRAL WEDGE

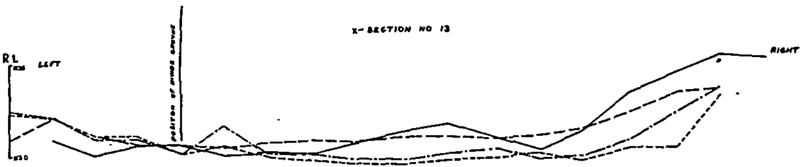


FIG. 97 CONTINUED

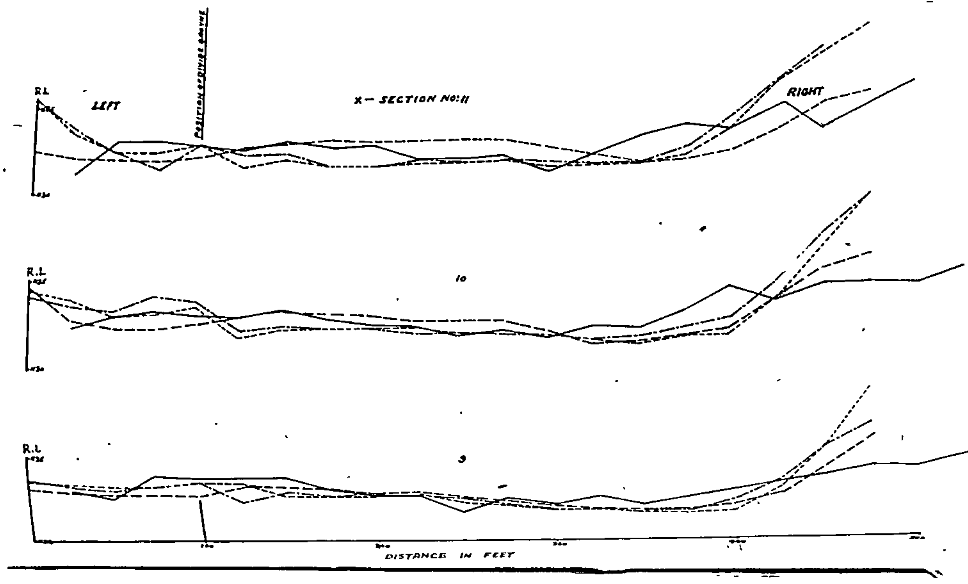
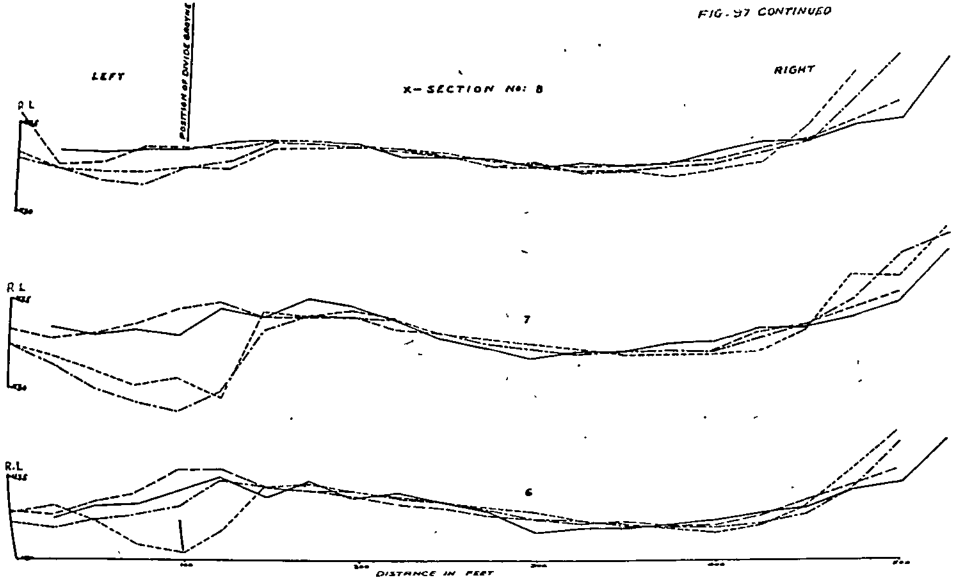
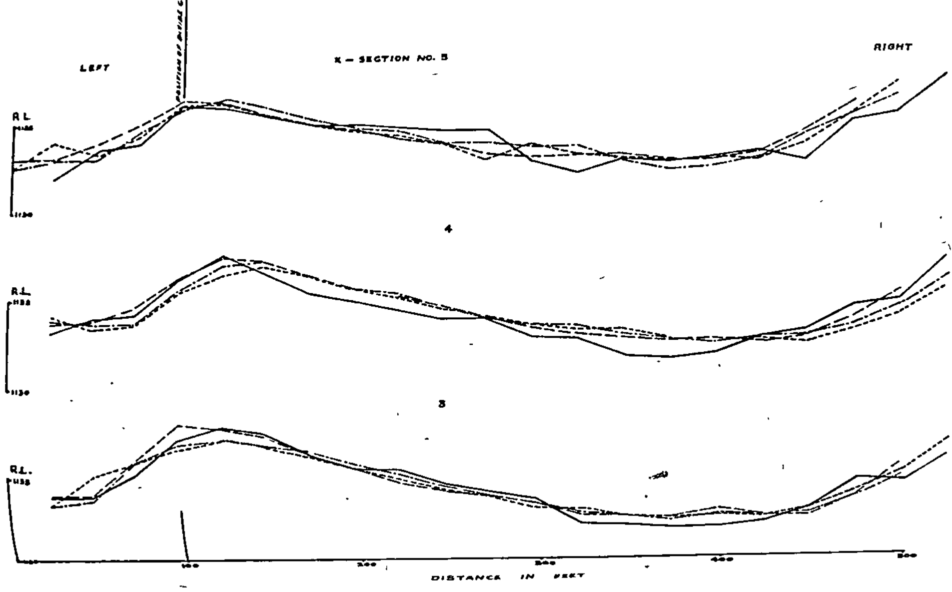
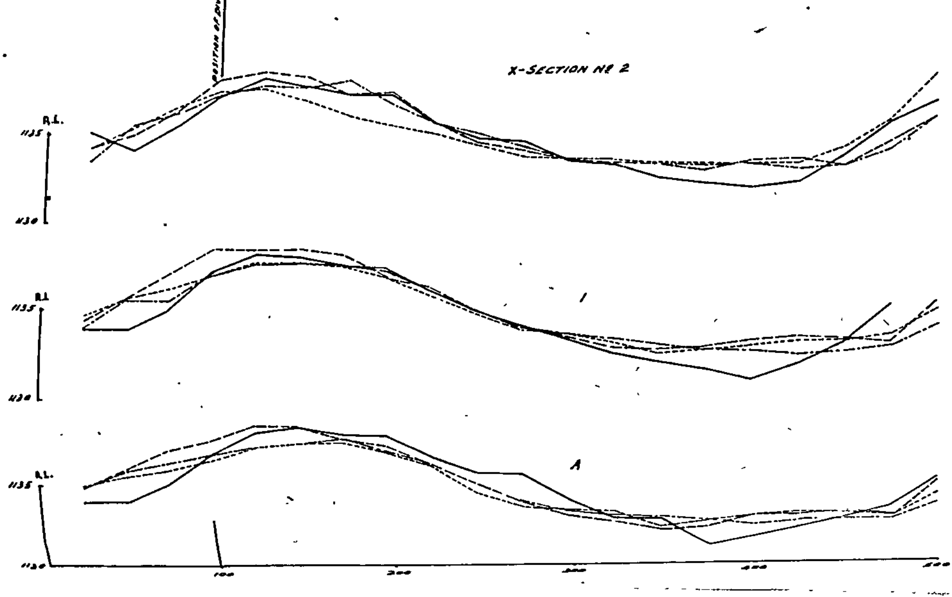


FIG. 97 CONTINUED

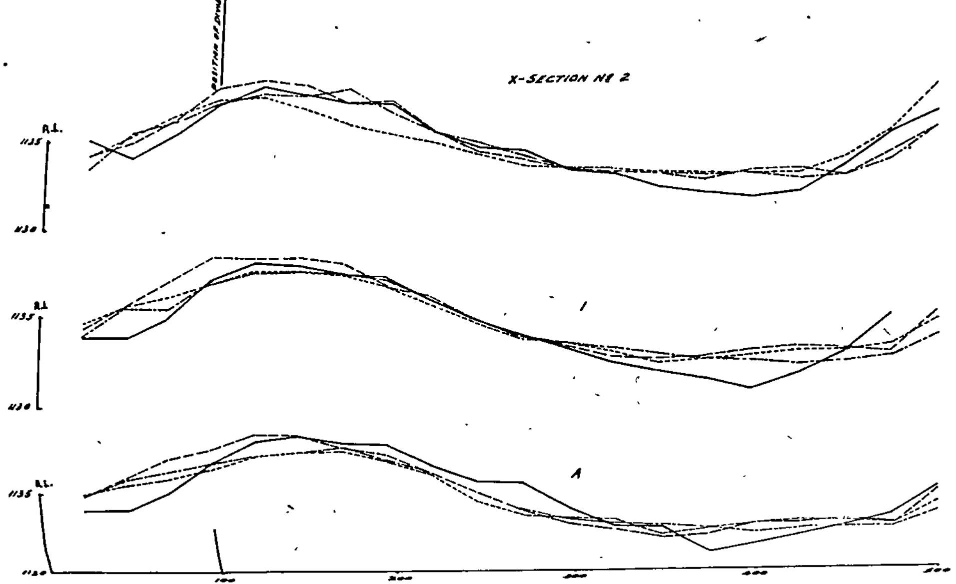


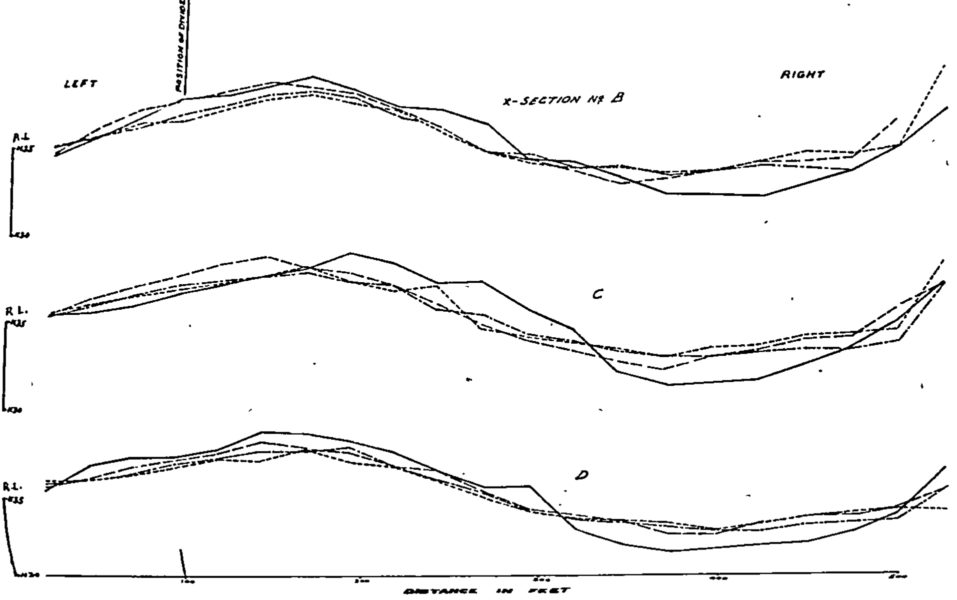


X – SECTION NO. 5
LEFT
RIGHT
R.L.
4
R.L.
3
R.L.
100
200
300
400
500
DISTANCE IN FEET



X-SECTION Nº 2
POSITION OF DIV
R.L.
1135
1130
R.L.
1135
1130
R.L.
1135
1130
I
A
100
200
300
400
500


X-SECTION No 2
R.L.
1135
1130
R.L.
1135
1130
R.L.
1135
1130
I
A
100
200
300
400
500


LEFT
POSITION OF DIVIDE
X-SECTION No B
RIGHT
R.L.
C
R.L.
D
R.L.
DISTANCE IN FEET

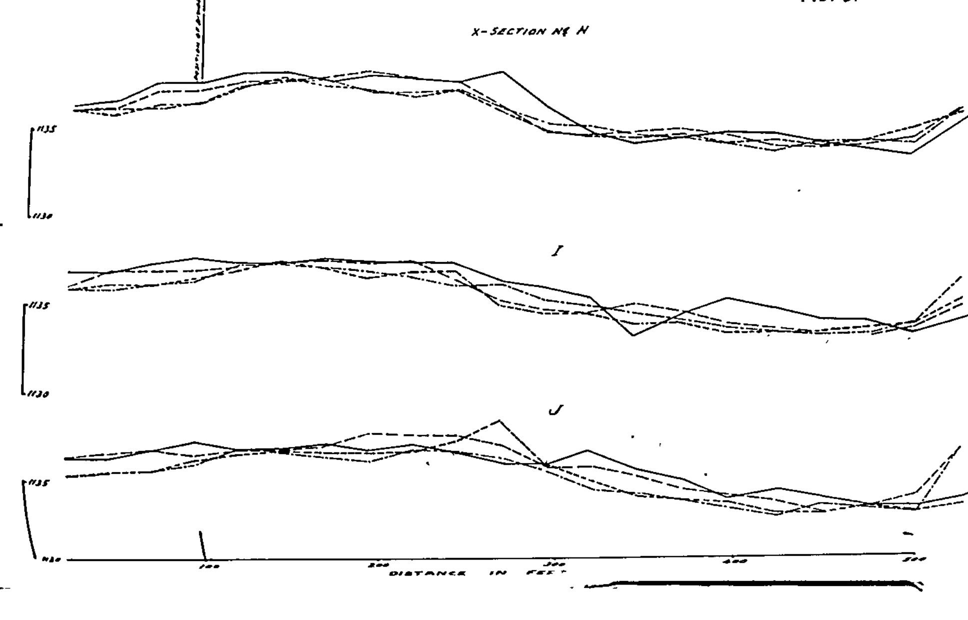
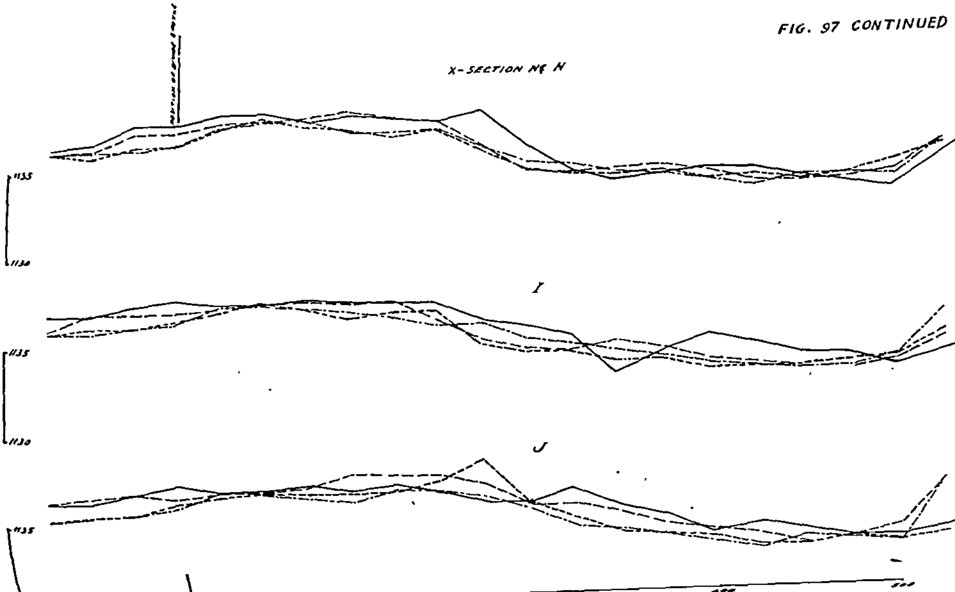
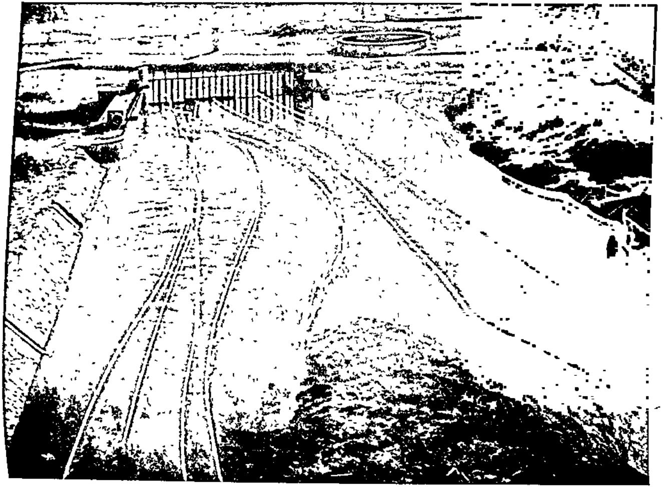

X-SECTION Nº H
I
J
1135
1130
1135
1130
1135
1130
100
200
300
400
500
DISTANCE IN FEET

FIG. 97 CONTINUED



MADHOPUR POCKET MODEL

Fig. 98



Photograph showing the current directions with divide wall at Pier No. 4

II. Testing the effect of the construction of spurs at the left edge of the main *bela*.

III. Testing the effect of raising the Shutters of the Supply weir.

I. *Testing the effect of construction of a divide groyne.*—A groyne 350′ long at R. L. 1,143 was constructed in the pocket at pier No. 4. The model was run with different discharges in the river to determine the conditions of flow with the groyne in position.

The following discharges were run :—

(1) 10,000 cusecs.
(2) 14,000 „
(3) 20,000 „
(4) 14,000 „
(5) 10,000 „
(6) 7,500 „

The different methods of regulation adopted were :—

(*i*) Wedge from the left, i.e., from Bays 1 to 12.

(*ii*) Still pond in the 1st four bays and wedge from the left in Bays 5—12.

(*iii*) Still pond in the 1st four bays and wedge from right in Bays 12—5.

At the end of the experiments river sections were taken and compared with the original levels. The model was laid to the original conditions for each test. A plot of these observations is given in Figs. 96 and 97. Silt entering the canal with different systems of regulation in the presence of the divide groyne was also measured. Results are given in Table XII.

The following conclusions were obtained :—

When water was escaped through Bays 1 to 4 in the presence of the divide groyne a huge deposit of silt occurred on the right side of the divide groyne. Scour, however, took place at the nose and a swirl formed on the left side. The *bela* was only slightly eroded.

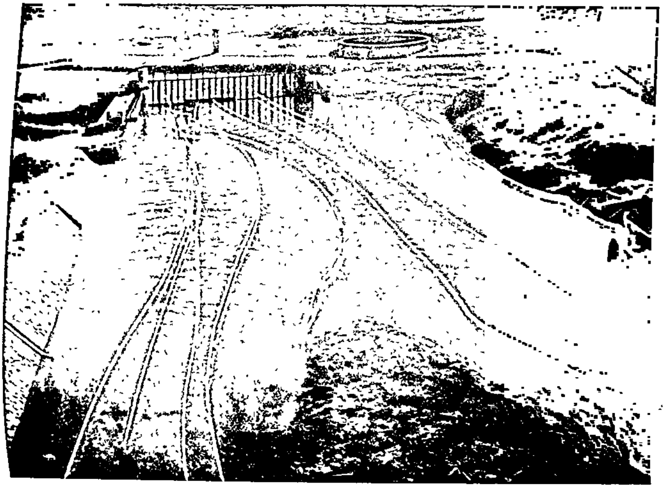
When the still pond system was adopted in Bays 1—4 a swirl formed at the nose of the divide wall resulting in a local scour. The *bela* was, however, not affected. More silt entered the canal, with the divide groyne in position. Current directions with divide groyne are given in Fig. 98.

It was concluded from this test that the construction of the divide groyne did not materially help in the removal of the *bela*.

II. *Testing the effect of construction of spurs on the left edge of the main bela.*—The model was again moulded to the original conditions

MADHOPUR POCKET MODEL

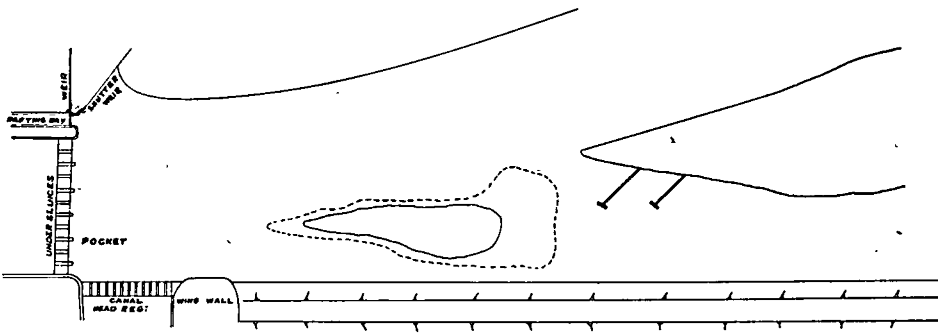
Fig. 98



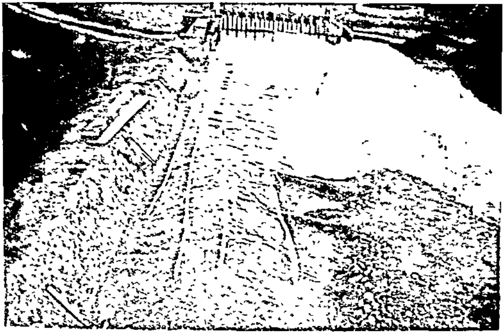
Photograph showing the [illegible]

FIG. 99

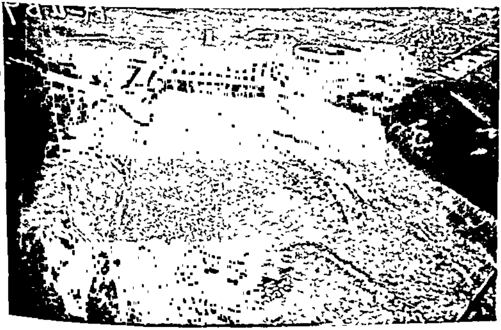
PLAN OF MADHOPUR POCKET MODEL



MADHOPUR POCKET MODEL
T Headed Spur



Wedge from Centre
Discharge : 10,000



After the run

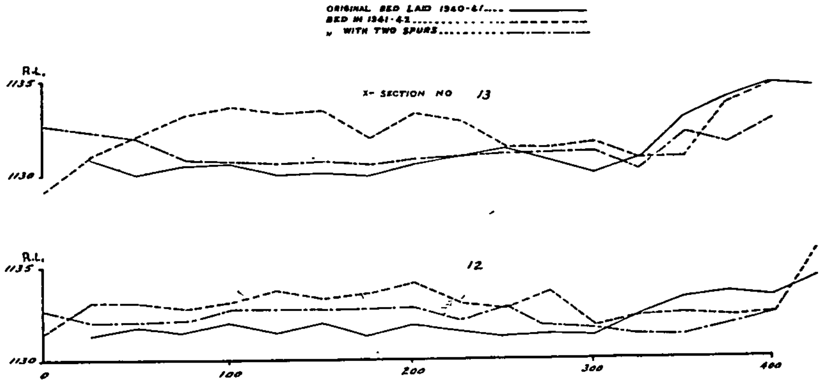
FIG. 101

MADHOPUR POCKET MODEL

X-SECTIONS SHOWING THE EFFECT OF TWO SPURS AT THE END OF PACCA BELA

U/S SPUR = 88 FT

D/S SPUR = 90 FT



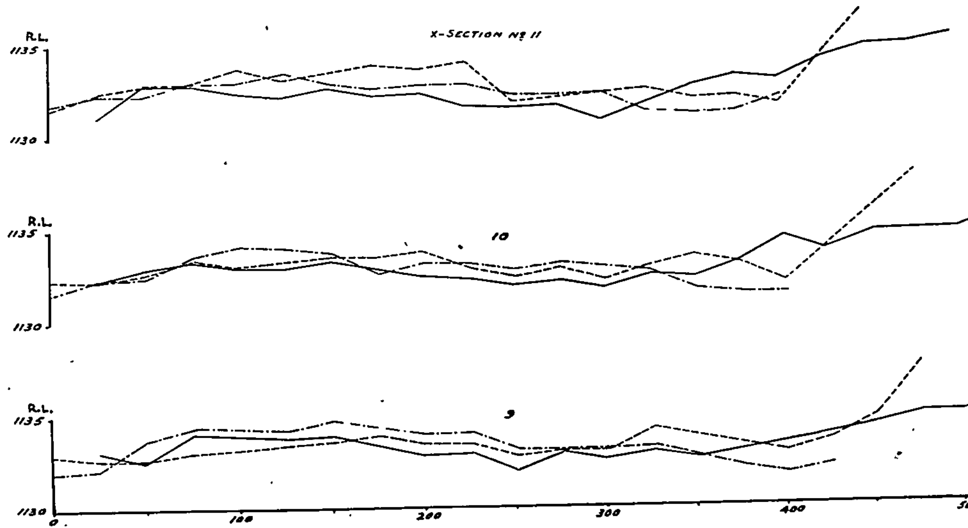

X-SECTION No 11
R.L.
1135
1130
10
R.L.
1135
1130
9
R.L.
1135
1130
0
100
200
300
400

FIG. 140 CONTI

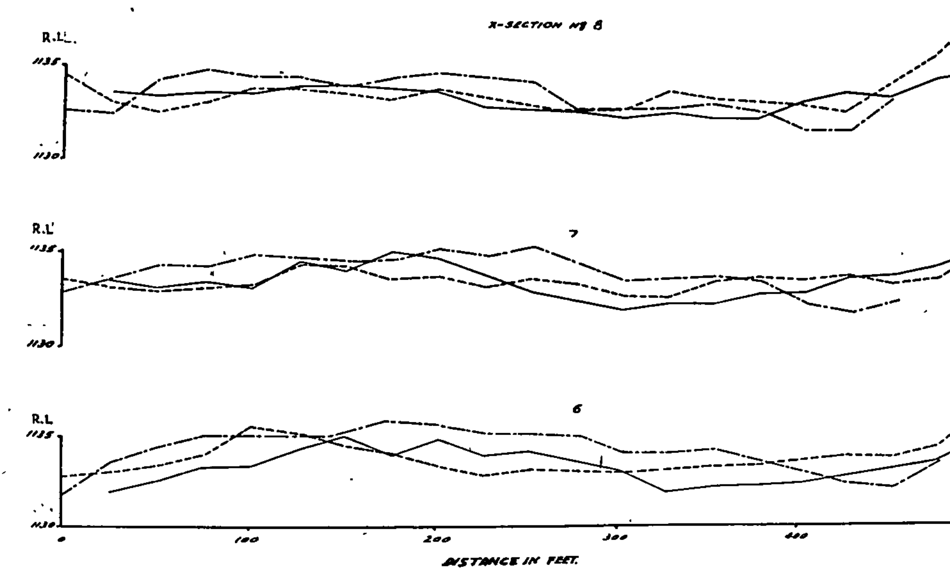
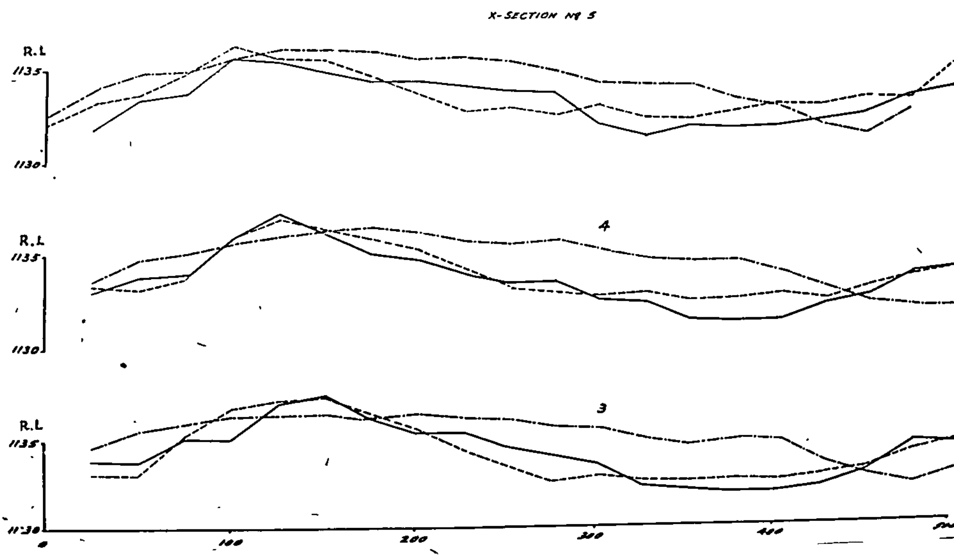


FIG. 140 CONTINUE



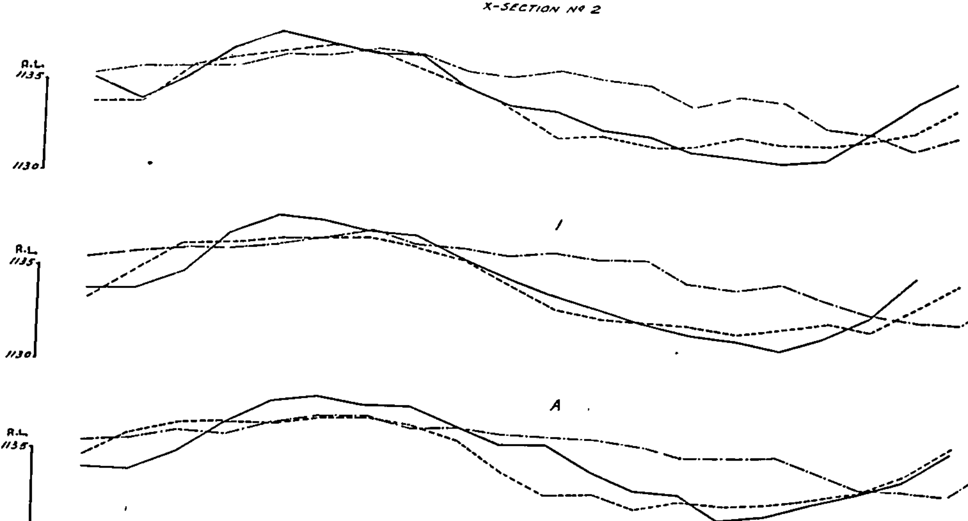

X-SECTION Nº 2
R.L.
1135
1130
1
R.L.
1135
1130
A
R.L.
1135

FIG 14 CONTINUED

X-SECTION NO B

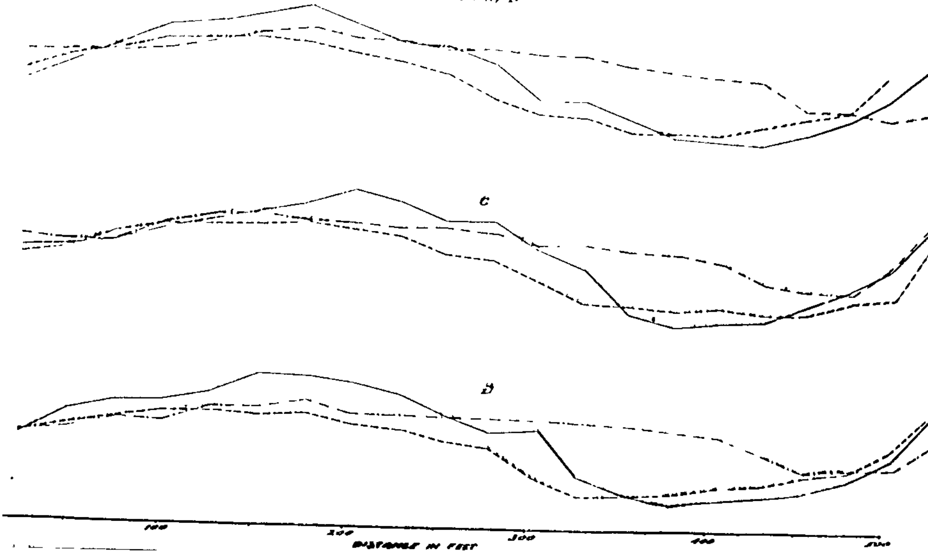
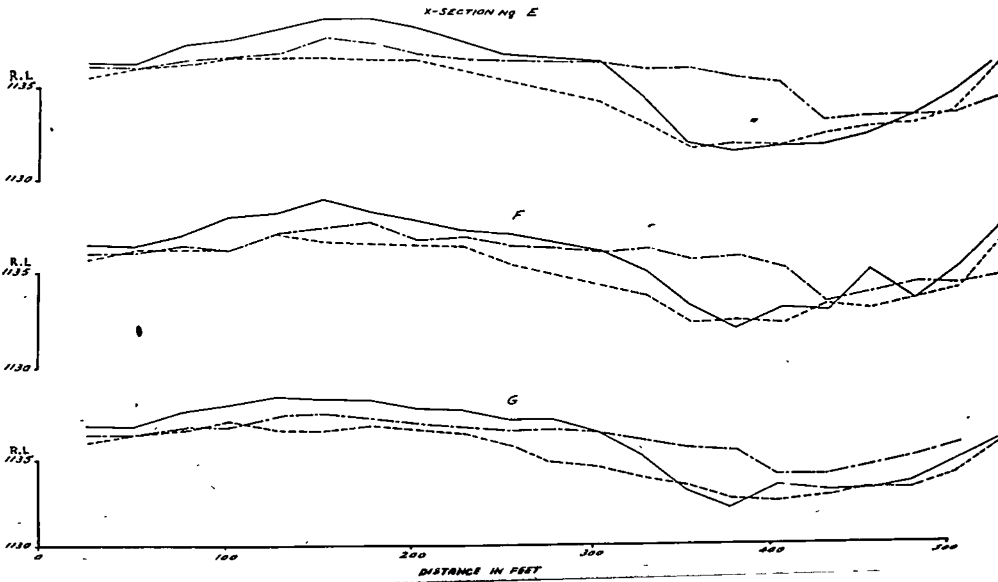


FIG 140 CONTINUED



X-SECTION Nº H

FIG 140 CONTINUED

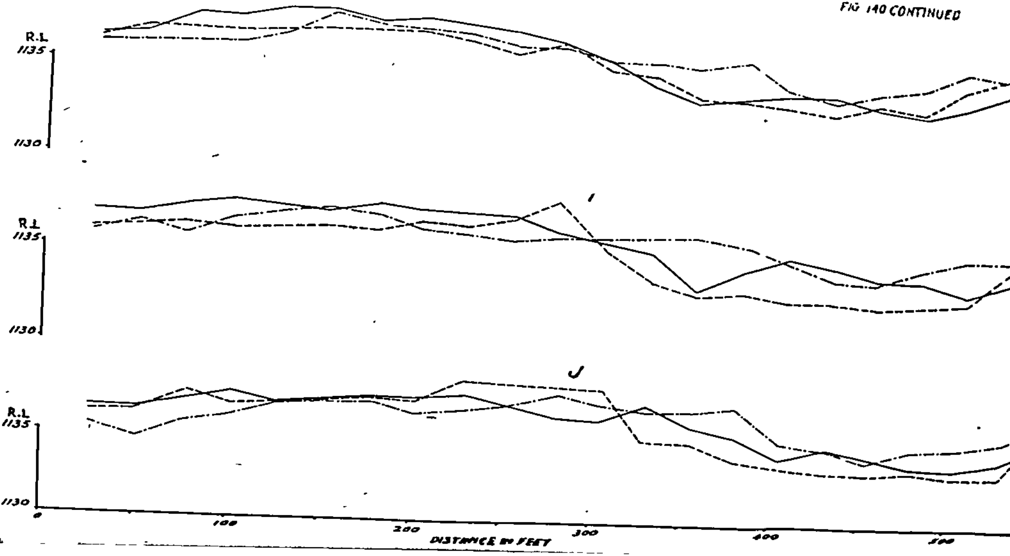
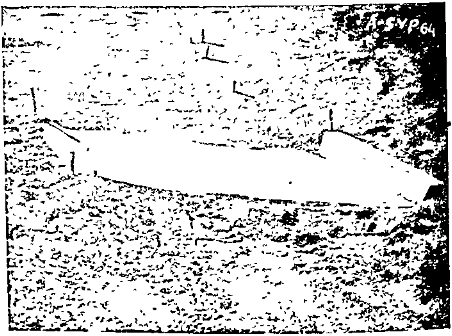


FIG. 104

PILOT MODEL OF RIVER SUTLEJ ABOVE PALLAH HEADWORKS



Showing the rigid section made at the upstream end of the model

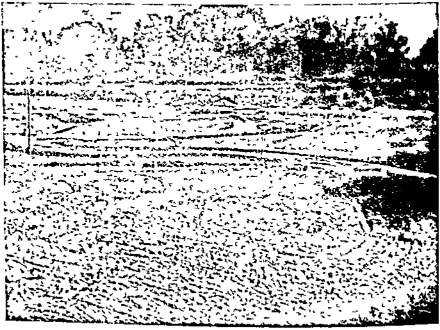
FIG. 105

PILOT MODEL OF RIVER SUTLEJ ABOVE PALLAH HEADWORKS

A view showing the training spurs and Headworks

FIG. 106

PILOT MODEL OF RIVER SUTLEJ ABOVE PALLAH HEADWORKS



Showing the complete view of the model and the loops

FIG. 107

PILOT MODEL OF RIVER SUTLEJ ABOVE PALLAH HEADWORKS

Showing the model running low discharge

The Chief Engineer ordered that model investigations should be made to determine the position and type of an armoured spur to protect the left bank against further erosion.

An examination of the River Survey Plans.—A study of the river course was made from the river survey plan of 1941. The river loop at R. D. 32,000 on the left retired embankment was examined and it was found that the loop formed a semi-circle with a radius of approximately 2,200 feet with more or less a fixed point at the Jhalar at line 65 of the cross section shown in Fig. 103.

An examination of the river upstream of this large loop showed that the reverse curve upstream of the loop had a radius of 8,000 feet.

Model investigation.—It was decided to construct a model of the river Sutlej for a length of 12 miles upstream of the Islam Headworks to horizontal and vertical scales of 1/100 and 1/20, respectively. As the land required for this model could not be obtained early it was decided by the Chief Engineer to first construct a pilot model and construct the large scale model after the land was acquired.

A small scale model was, therefore, first constructed in the tray occupied by the Khanki Headworks model. The scales adopted were :—

Horizontal	1/300
Vertical	1/50
Exaggeration	6 times.

The levels of inlet and exit in this model were fixed in such a way that water could escape without dismantling the Khanki Weir. A rigid section was made at the upstream end and is shown in Fig. 104. A view of the training works and the river loops are shown in Figs. 105 and 106, respectively. A view of the model running with the minimum winter conditions is shown in Fig. 107.

The model could not be checked as no previous complete survey of this reach was available. The first experiment consisted in testing the model to determine the conditions of the river edge in 1942. The model was run for the discharges similar to those experienced in 1941. From this model it was shown that very little bed movement and bank erosion occurred in the loop at R. D. 32,000 of the left retired embankment. However, above and below the loop there were considerable changes in the bed. An examination of the model in medium discharges showed that the velocity of water in the loop, i.e., between sections 65 and 51 (Fig. 103) was very low while it was high upstream and downstream of the loop. An indication obtained from this test was that there may not be further shifting of the left bank. However this could not be ascertained definitely from the small scale model and required confirmation of the large scale model.

Construction of Spurs.—Next the possibility of the construction of the spur in order to protect the left bank was examined. It was

shown that at least two armoured spurs were required to protect the left bank. The river course may be fixed in that loop but the maintenance of the spur would cause heavy expenditure.

An alternative scheme of cutting off the loop by constructing a bund in the main channel and diverting the river was next examined. The following proposals were made for this investigation :—

(1) *Constructing a bund between sections 49 and 50 and making a leading cut from sections 47-L to 74-L.*—In Fig. 103 there is an old creek marked A. It was first suggested to utilize this creek for making the leading cut. However the Overseer from Islam stated that at site the creek had almost silted up and that there was considerable growth of *pilchhi* on it. It was also mentioned by him that now there was no difference in the levels of the *bela* and that of the creek. A cut was, therefore, proposed between sections 47-L and 74-L.

(2) *Constructing a bund between sections 50 and 51 and making a leading cut from sections 48-L to 72-L.*—This was an alternative proposal of shifting the cut slightly upstream.

(3) *Constructing a bund between sections 53 and 54 and making a leading cut from section 43 to section 81·5.*—The last proposal was examined first as it had the advantage of straightening out the river. The cut developed very little. It was decided by the Chief Engineer to obtain a detailed section from the Executive Engineer, Islam, of this area before carrying out further experiments.

VII—An investigation of a model of the River Jumna above Tajewala Headworks.

A model of the River Jumna for a length of three miles upstream of the Western Jumna Canal Head Regulator and about a mile downstream of the regulator was constructed at Malikpur to the following scales :—

Horizontal	1/40
Vertical	1/12
Distortion	3·33

The bed material used in the model varied from ¼" to ¾" in the main stream and 1" to 1½" in *belas*. This was done after a number of preliminary experiments. A view of the model at different points is shown in Figs. 108, 109 and 110.

After constructing the model, tests were carried out on the effect of shortening the length of spur No. 6. The length of the spur was reduced first by 25' and then by 50' and the directions of flow were observed. The model showed that no useful purpose could be served by reducing the length of spur No. 6. The spur was then extended into the river by about 100 feet and the effect determined. It was shown that lengthening the spur had the effect of bringing more water on to the left side.

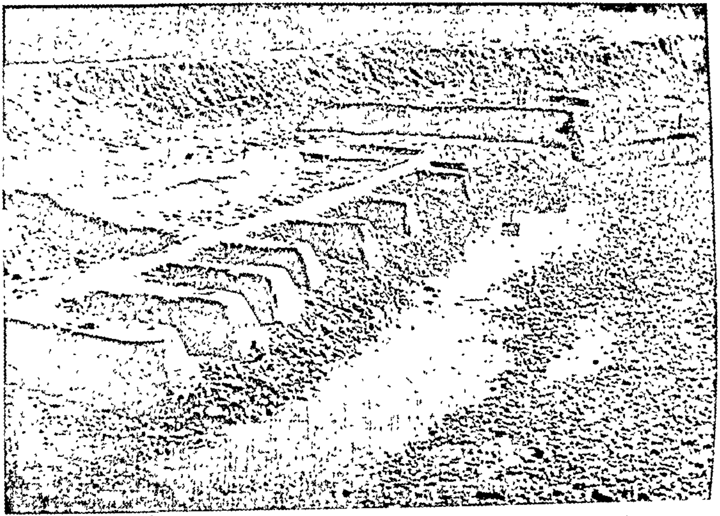
Fig. 10

MODEL OF RIVER JUMNA ABOVE TAJEWALA HEADWORKS

A view of the Johnson Nose, Gordan groyne and the Block-bar spur

FIG. 109

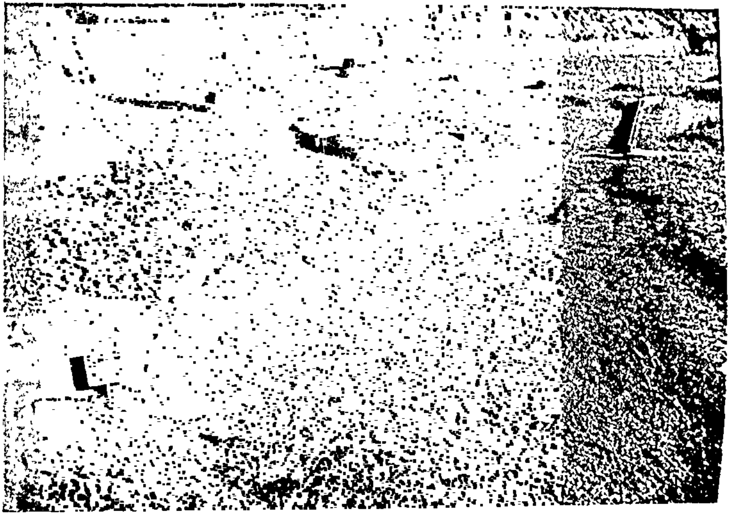
MODEL OF RIVER JUMNA ABOVE TAJEWALA HEADWORKS



A view of the series of spurs along the left bank upstream of Spur No. 6

FIG. 110

MODEL OF RIVER JUMNA ABOVE TAJEWALA HEADWORKS



Photograph showing the New Weir and Spur No. 6

5. Next the effect of construction of a shingle bar upstream of the Block Bar Spur and in front of the old weir was examined. It was shown that with the construction of the shingle bar most of the shingle and sand could be diverted to the old weir. The most suitable position of the bar is being examined.

B. MODELS OF FALLS AND RAPIDS.

The following investigations were carried out in this connection :—

I. Further investigations of models of Rapids on the Western Jumna Canal.

II. An investigation of models of the Fall at R. D. 217,000, Dipalpur Canal to examine methods of protection of downstream bed and sides.

III. An investigation of a model of Rapid at R. D. 12,500, Lahore Branch.

IV. An investigation of a model of the Ralliali Rapid at R. D. 11,900 Main Branch Lower, Upper Bari Doab Canal.

V. An investigation of a model of V. R. Bridge at R. D. 40,510, Lower Jhelum Canal.

I.—Further investigations of models of Rapids on the Western Jumna Canal.

A report on the results of experiments carried out on different types of rapids to be constructed on the Western Jumna Canal, was submitted to the Chief Engineer in the Annual Report of the Research Institute for the year ending April, 1941. It was shown in the report that the fall-*cum*-rapid design was the most suitable of all the types examined.

The Chief Engineer inspected the models of rapids on the 19th of August 1941 and examined the conditions of flow in each case for varying discharges in the canal with special reference to the difficulties in the raft traffic.

After inspection, the Chief Engineer ordered that further experiments on the models of rapids should be carried out to determine the necessary alterations in design in order to facilitate the passage of rafts over the rapids. He desired that the glacis slope used in the Rafting Bay at Madhopur Headworks should be examined.

The Rafting Bay at Madhopur has a glacis on a slope of 1 in 40. The glacis slope of 1 in 40 was, therefore, examined on these rapids.

Tests were carried out on the rapid portion of the fall-*cum*-rapid design. The original slope of 1 in 15 was converted to 1 in 40. The model was run with different discharges varying from 2,000 to 9,000 cusecs and observations were made for the position and type of standing wave and scour downstream.

From the type of standing wave formed, indications could be obtained for the suitability or otherwise of the design for the raft

traffic. The standing wave which was not well defined and consisted of only surface ripples afforded an easy passage to the rafts.

Tests were also made in each case to determine the facility of raft traffic on the altered glacis slope. On the model, the rafts were represented by wooden pieces 8 to 9 inches long, 1·5″×1″ and tied together. These were let into the canal upstream of the crest of the rapids and their course was determined. The conditions of flow in which all the rafts passed through the standing wave freely were considered to be the most suitable.

It must be stated that the tests w ... and comparative and give just a roug... the rafts. All the rafts on the prototype may not behave exactly as they do on the model. Even on the prototype the behaviour of the rafts is not uniform. Some rafts pass through the standing wave while others, under similar conditions, get stuck in.

The results obtained with different discharges are discussed in the following pages. Photographs illustrating the conditions of flow in each case are also given.

In order to compa ... ...ditions of flow on the new glacis slo... model, photographs were taken with similar discharg...

2,000 *cusecs dischar*... represent the minimum w... of flow on the rapids in tl... be seen from this figure th... at 157 feet downstream of the crest. The position ... is not fixed and moves up and down. It mostly consists of a forward moving jet. All the rafts which floated upstream of the rapids passed through the standing wave without the least hinderance. No scour took place downstream of the work. The conditions of flow in 1 in 15 glacis slope are given in Fig. 112.

3,000 *cusecs discharge in the Canal.*—The conditions of flow in this discharge are illustrated in Fig. 113. There is not much difference between this case and the previous one. The conditions of flow in 1 in 15 glacis slope are given in Fig. 114.

4,000 *cusecs discharge in the Canal.*—The discharge in the canal was further increased to 4,000 cusecs. The conditions of flow obtained in this case are shown in Fig. 115. It will be seen from an examination of this figure that in this case also no proper standing wave forms. All the rafts dropped into the water got through the wave without stopping.

... *the Canal.*—The conditions of flow in ... slope of 1 in 40 are ... ,000 cusecs discharge. The conditions are ... the conditions of flow in 1 in 15 glacis slop... No scour occurred downstream of the w...

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis slope 1 in 40

Discharge 2,000 cusecs

FIG. 112

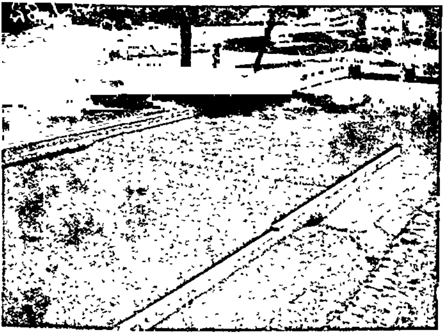
MODEL OF A RAPID ON WESTERN JUMNA CANAL
Glacis slope 1 in 15

Discharge 2,000 cusecs

FIG. 112

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis slope 1 in 15

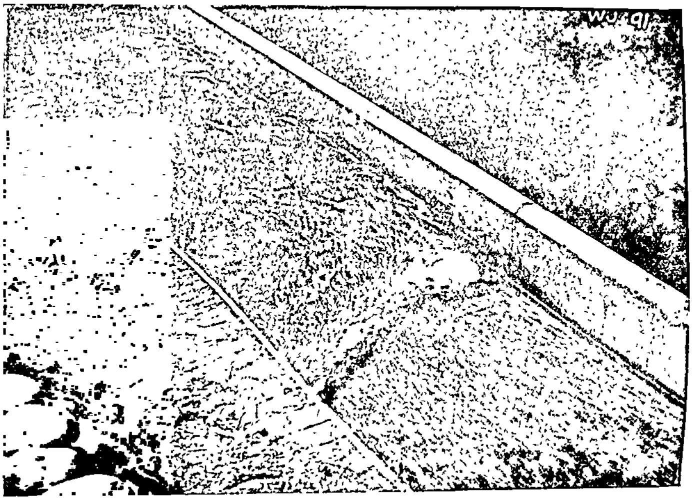


Discharge 2,000 cusecs

FIG. 113

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 40

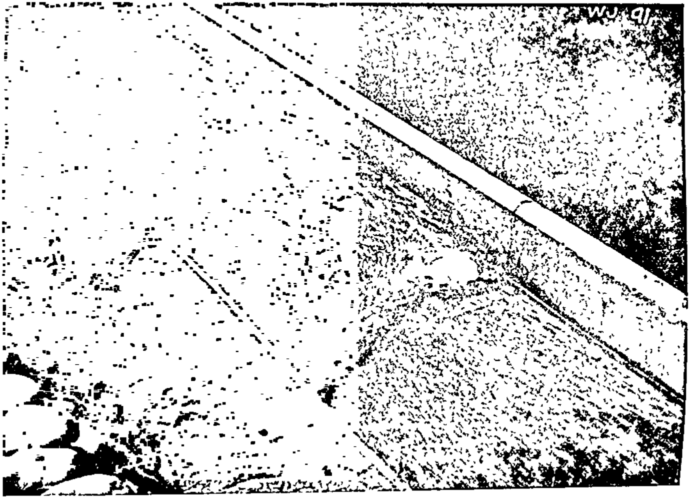


Discharge 3,000 cusecs

FIG. 113

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 40

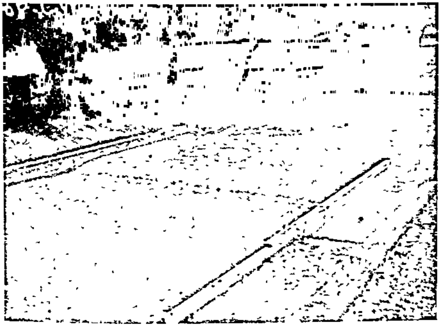


Discharge 3,000 cusecs

FIG 114

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 15

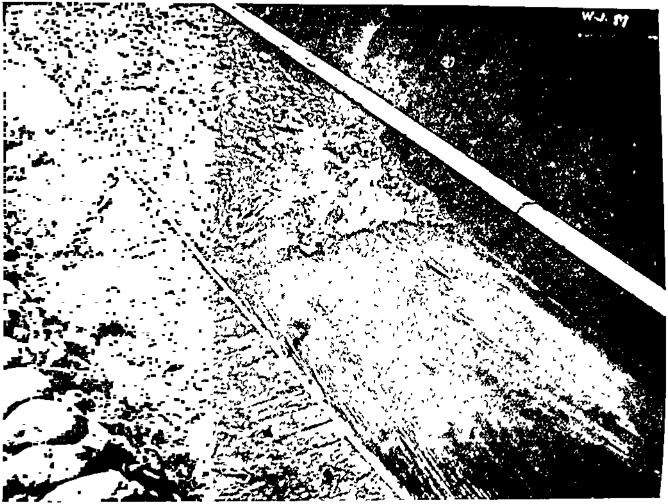


Discharge 3,000 cusecs

FIG. 115

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 40



Discharge 4,000 cusecs

FIG. 116

MODEL OF A RAPID ON WESTERN JUMNA CANAL

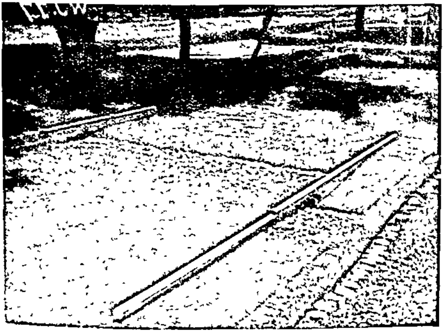
Glacis Slope 1 in 40

Discharge 5,000 cusecs

Fig. 117

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 15



Discharge 5,000 cusecs

Fig. 118

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 40

Discharge 7,000 cusecs

FIG. 119

MODEL OF A RAPID ON WESTERN JUMNA CANAL

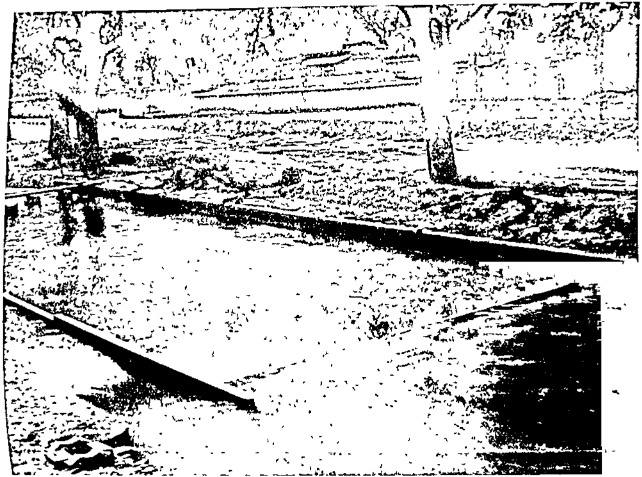
Glacis Slope 1 in 40

Discharge 9,000 cusecs

FIG. 120

MODEL OF A RAPID ON WESTERN JUMNA CANAL

Glacis Slope 1 in 15



Discharge 9,000 cusecs

FIG. 121

1/20 SCALE TEAKWOOK MODEL OF V.R. BRIDGE AND FALL AT R D 217,000 DIPALPUR CANAL

SCALE = 1/10

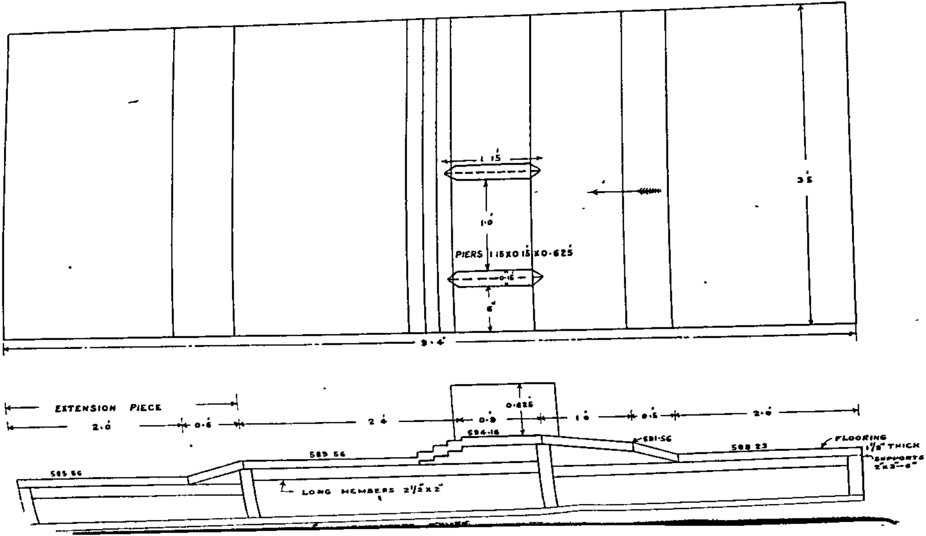
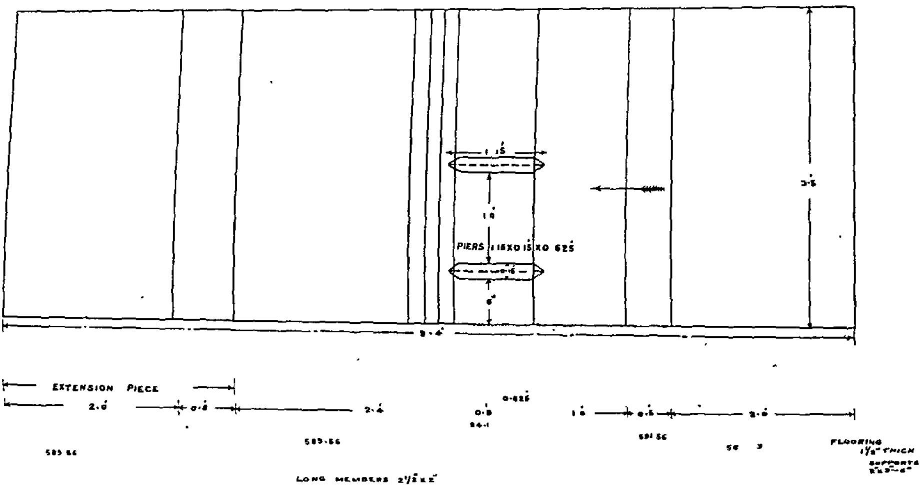


FIG. 121

1/20 SCALE TEAKWOOD MODEL OF V.R BRIDGE AND FALL AT R D 217000 DIPALPUR CANAL

SCALE = 1/10



7,000 *cusecs discharge in the Canal.*—The conditions of flow observed in this discharge are illustrated in Fig. 118. No standing wave formed in the portion from the left bank to a point 20 feet from the left bank, and a very poor wave occurred in the remaining 10 feet. One out of six rafts floated upstream got stuck in the standing wave. The standing wave shifted from a position 162 feet to 175 feet downstream of the crest. The depth of scour in this case was about 0·8 feet. It must be stated that on the model the coarse sand formed the bed of the canal downstream of the impervious floor. Since, at site, shingle exists there would be no scour at all.

Full supply conditions in the canal.—*9,000 cusecs discharge.*

The conditions of flow in the full supply discharge on the altered glacis slope are illustrated in Fig. 119. The position of the wave shifted from 118 feet to a point 184 feet downstream of the crest. ometimes it was formed but more mostly got through the standing the 1 in 15 glacis slope are given in Fig. 120.

An important indication obtained from the above observations that with the alteration in the glacis slope from 1 in 15 to 1 in 40 e difficulty in the raft traffic may decrease considerably. It may not necessary to alter the design in the entire width of the rapid ; only to 30 feet width may be reconditioned which may be sufficient for e passage of the rafts.

In another series of experiments the crest of the rapid was lowered the level of the upstream floor, i.e., R. L. 92·75 and experiments epeated. It was shown that the conditions of flow in this case were ightly more suitable for passage of rafts than those obtained with the aised crest.

II—An Investigation of models of the Fall at R. D. 217,000, Dipalpur Canal to examine methods of protection of downstream bed and sides.

The experiments were carried out in accordance with the Chief Engineer's orders contained in his letter No. 7294-N/506/35, dated the 17th April 1941.

Design of the Fall—The design of the fall at R. D. 217,000 Dipalpur Canal is illustrated in Fig. 121. The drop between the crest and the downstream floor is 4·60 feet and is secured in four steps each 1·15 feet high. At the downstream end of the horizontal floor there is a baffle wall two feet high. Downstream of the baffle wall there is a sloping glacis and a horizontal dry brick apron.

Considerable action takes place on the bed as well as at the sides.

Examination of models—Two models of this fall were examined :—

(1) A model representing a section of the fall only.

(2) A model representing a section and a side of the fall.

The models were made in seasoned teak-wood with horizontal and vertical scales of 1/20. The conditions of flow examined were as follows :—

(a) Full supply conditions—

U/S water level	..	R. L. 600·06
D/S water level	..	R. L. 598·06
Discharge	..	2,235 cusecs.

(b) Half supply conditions—

U/S water level	..	R. L. 598·2
D/S water level	..	R. L. 597·9
Discharge	..	1117·5 cusecs.

(c) Quarter supply conditions—

U/S water level	..	R. L. 596·8
D/S water level	.. .. ..	R. L. 595·3
Discharge	.. .. ..	558·8 cusecs.

(d) Conditions of flow over the fall when the canal is just opened—

U/S water level	.. .. ..	R. L. 596·0
D/S water level	.. .. ..	Just dry
Discharge	.. .. ..	Between 400 & 500 cusecs.

Investigation of Model No. 1—Representing a section of the Fall only.

This model was examined first for all the conditions of flow. The baffle wall, which is partly broken, was not represented on this model. A plot of the observations is given in Figs. 122 and 123. From an examination of these figures it will be seen that no appreciable scour occurs on the bed on this model under any conditions of flow tested above.

In the next series of tests on this model the downstream water level was lowered firstly by one foot and then by two feet below the normal water level in order to stud[illegible] the downstream level on the action [illegible] obtained are plotted in Figs. 124—127. [illegible] retrogressed some action did occur on the bed but even under these conditions the maximum scour on the bed was only 3·92 feet.

One of the important indications obtained from these test was that the *design of the section of the work mainly is not responsible for the scour on the bed.*

FIG. 122

DIPALPUR CANAL FALL AT R.D. 217,000

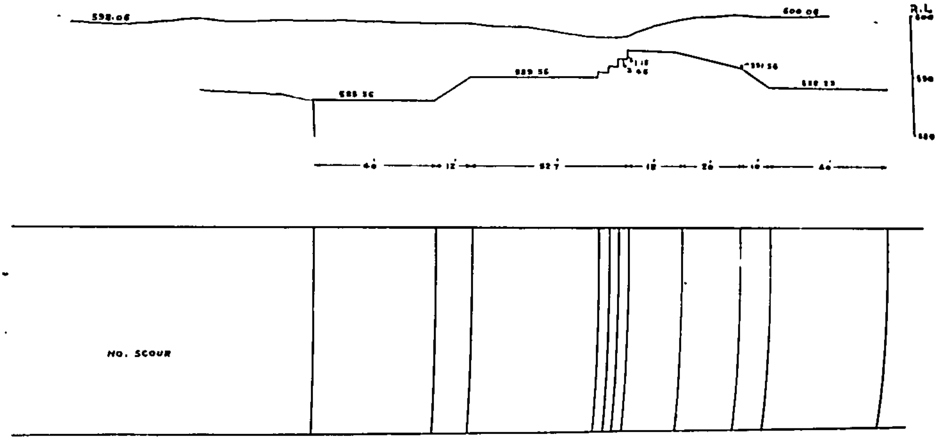


FIG. 123

FALL AT R.D. 2,17,000 DIPALPUR CANAL

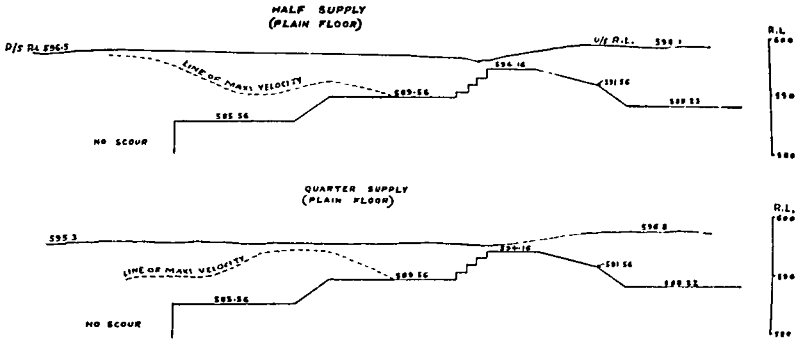


FIG. 124

FALL AT R.D. 2,17,000 DIPALPUR CANAL

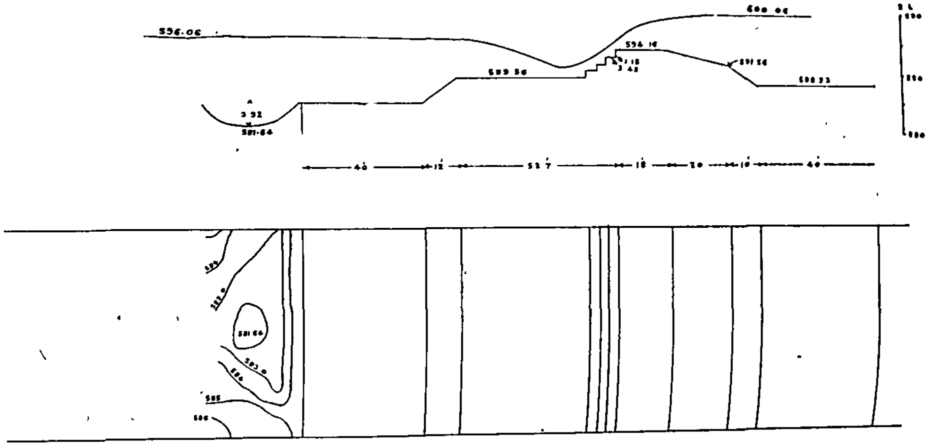


FIG. 125

FALL AT R. D. 2,17,000 DIPALPUR CANAL

FULL SUPPLY

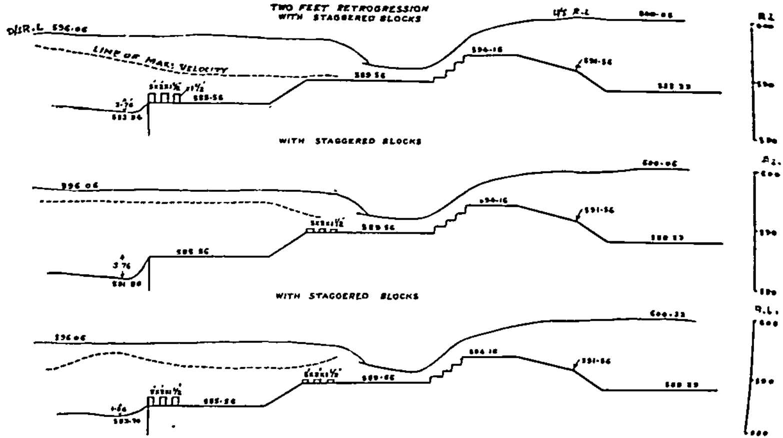


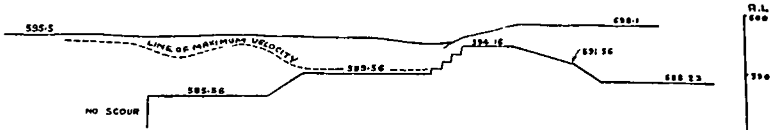
FIG 126

FALL AT R.D. 217,000 DIPALPUR CANAL

HALF SUPPLY

ONE FOOT RETROGRESSION

PLAIN FLOOR



FULL SUPPLY

TWO FEET RETROGRESSION

WITH STAGGERED BLOCKS

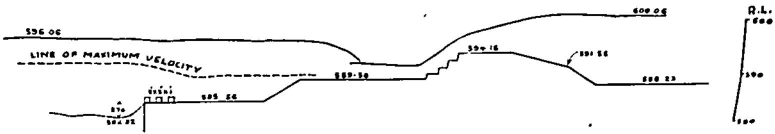


FIG. 127

FALL AT R.D. 217,000 DIPALPUR CANAL

FULL SUPPLY

TWO FEET RETROGRESSION

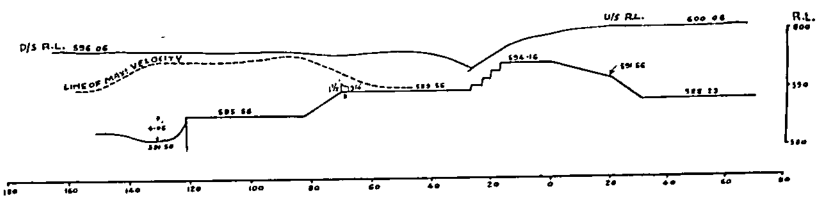


FIG. 128

DIPALPUR CANAL FALL AT R D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

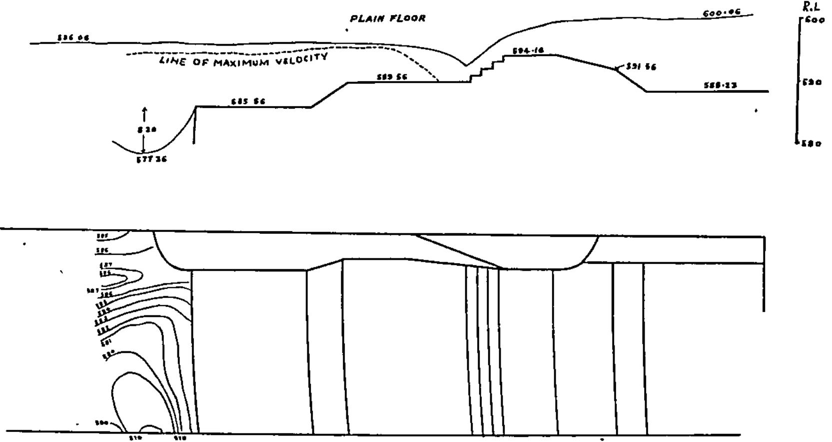


FIG. 129
DIPALPUR CANAL FALL AT R.D. 217,000
PARTIAL MODEL
FULL SUPPLY

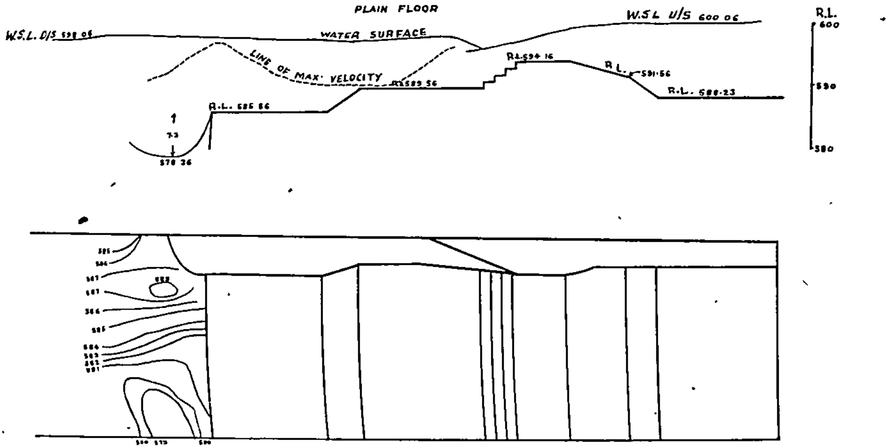


FIG. 130

DIPALPUR CANAL FALL AT R.D. 2.17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

CONDITION IN THE PRESENCE OF A BAFFLE WALL

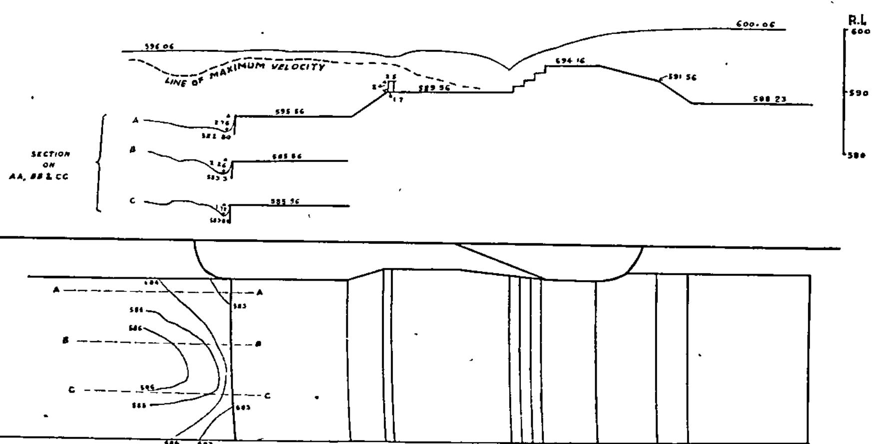


FIG. 131

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL
FULL SUPPLY

CONDITION IN THE PRESENCE OF A BAFFLE WALL

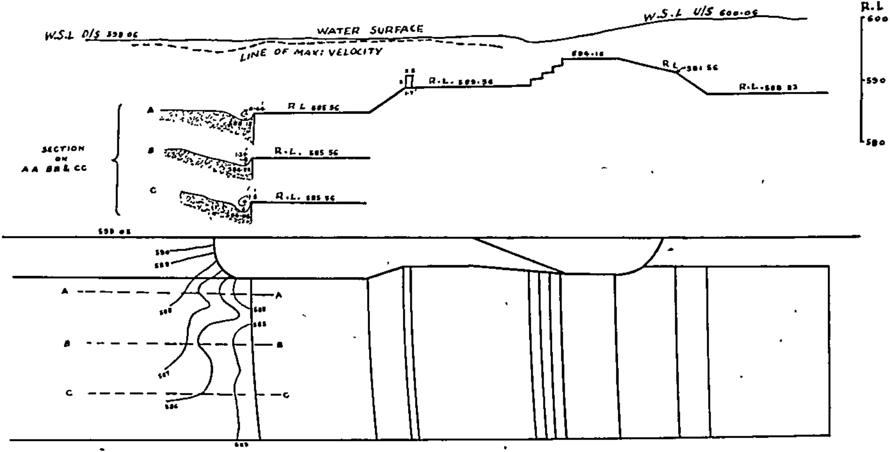


FIG. 131

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

CONDITION IN THE PRESENCE OF A BAFFLE WALL

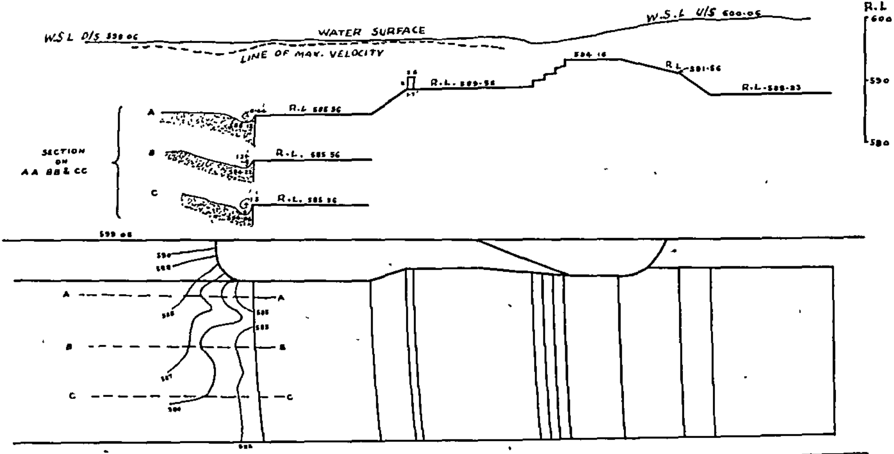


FIG. 132

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

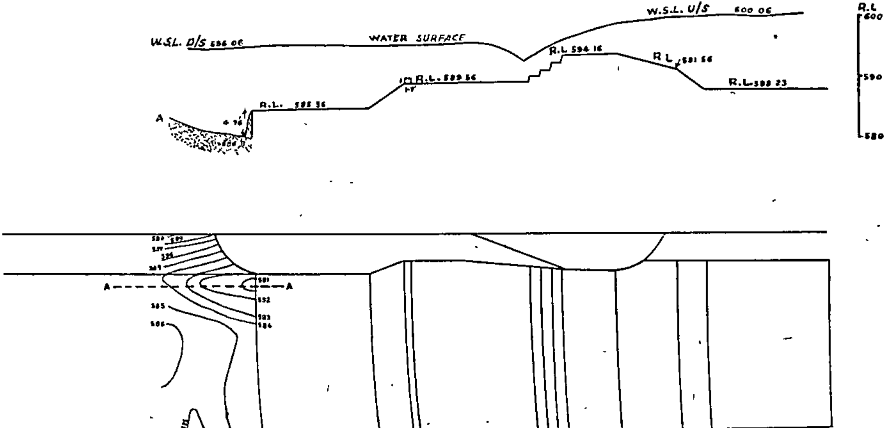


FIG. 132

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

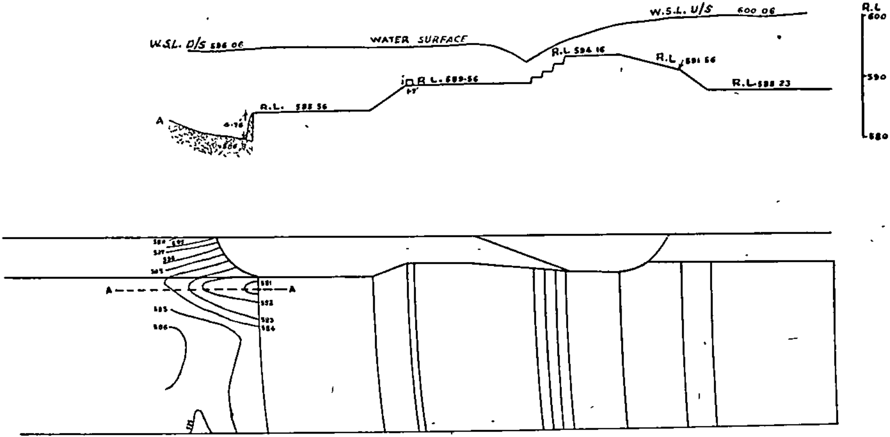
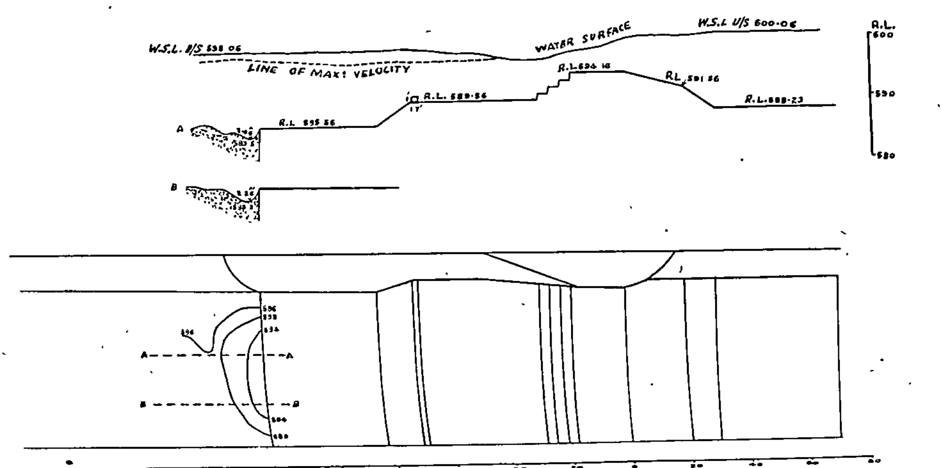


FIG. 133

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY



Investigation of Model No. 2—Representing a section and a side of the canal.

On this model the right side was also represented and the required fluming at the crest was given. The sides below the pacca pitching were made with a mixture of sand and clay in the ratio of 1 : 1.

The worst conditions of flow, i.e., full supply levels with 2 feet retrogression on the downstream water level was first examined. All the necessary observations were taken and are plotted in Fig. 128. From an examination of this figure it will be seen that the bed scour equivalent to 8·2 feet occurred on this model. On the first model the depth of scour under similar conditions was 3·92 feet only.

Full supply conditions with the normal levels upstream and downstream were next examined. A plot of the observation is given in Fig. 129. Scour 7·8 feet deep was obtained with the normal levels. The depth of scour obtained in the previous model under similar conditions of flow (Fig. 122) was nil. *This clearly indicates that the deep scour holes in the bed are due to the fluming at the crest and the defective divergence downstream.*

In the next series of tests a complete baffle wall was constructed at a position similar to that at site and the model was again run. The following conditions of flow were examined :—

(i) Full supply conditions with the downstream water level retrogressed by 2 feet.

(ii) Full supply conditions with normal water levels upstream and downstream.

The observations for the bed scour, line of maximum velocity and the water surface profile are plotted in Figs. 130—131. As the scour obtained in the bed in this case was not uniform, contours of the scoured bed were taken and are also plotted in the above figures. As a result of the study of this model the following indications were obtained :—

(1) Downstream of the baffle wall the conditions of flow were not uniform. There was a jetting action at the side and a secondary jet occurred in the centre. This action originated at the baffle wall.

(2) The side scour was very much pronounced.

(3) The scour on the bed, however, was much less than that obtained when the baffle wall was absent.

[illegible] of the baffle wall another [illegible] of the baffle was reduced [illegible] feet to 1 foot. The observations obtained for these conditions are plotted in Figs. 132-133. From an examination of these figures it [illegible] of the baffle wall increased [illegible] reduced.

From a study of these observations it can be concluded that the baffle wall though reducing the bed scour, creates a jet [illegible]

Fig. 135

DIPALPUR CANAL FALL AT R. D. 217,000

Floor with blocks

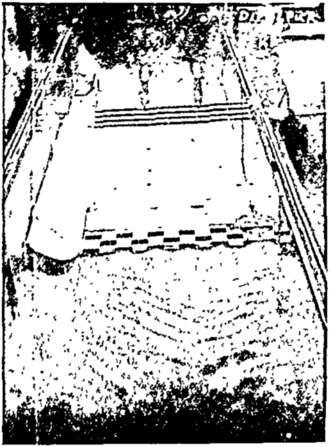
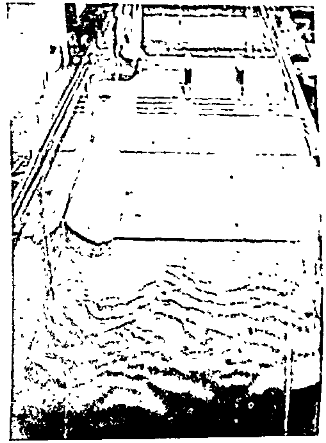


Fig. [illegible]

DIPALPUR CANAL FALL AT R. D. 217,000

Floor with Baffle Wall



Model with the baffle wall one foot high

FIG. 185

DIPALPUR CANAL FALL AT R. D. 217,000

Floor with blocks

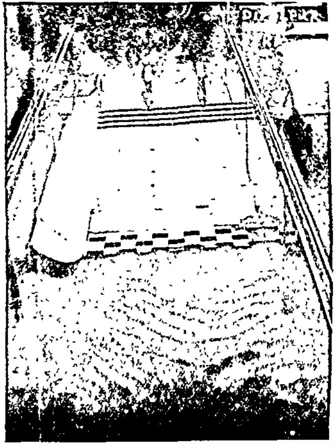
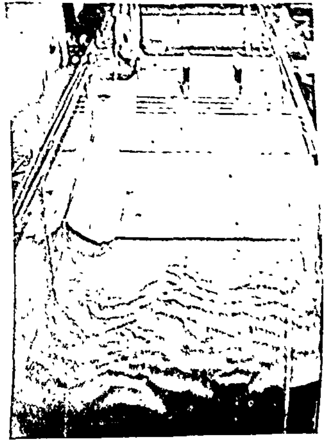


FIG. 1[illegible]

DIPALPUR CANAL FALL AT R. D. 217,000

Floor with Baffle Wall



Model with the baffle wall one foot high

Fig. 137

DIPALPUR CANAL FALL AT R. D. 217,000

Plain Floor

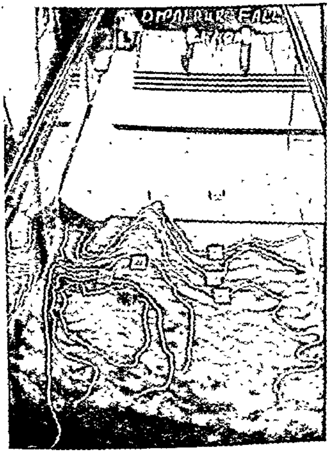


FIG. 138

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

STAGGERED BLOCKS AT THE END OF THE FLOOR

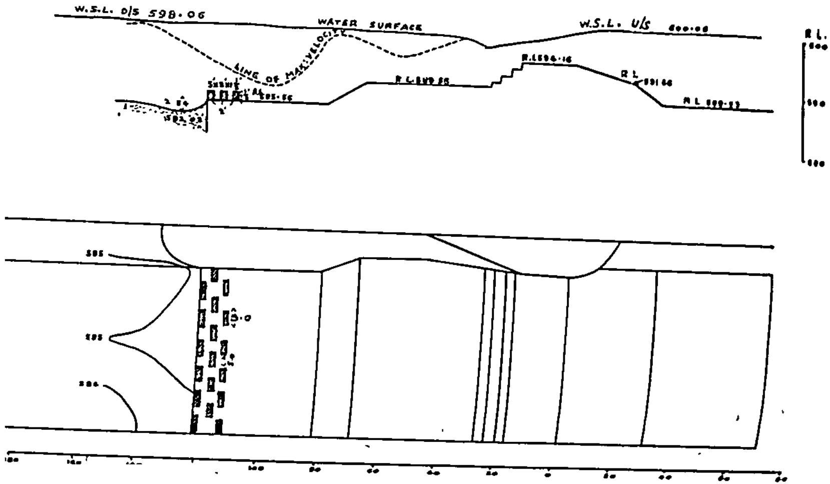


FIG. 139

DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

WITH STAGGERED BLOCKS AT THE END OF THE FLOOR

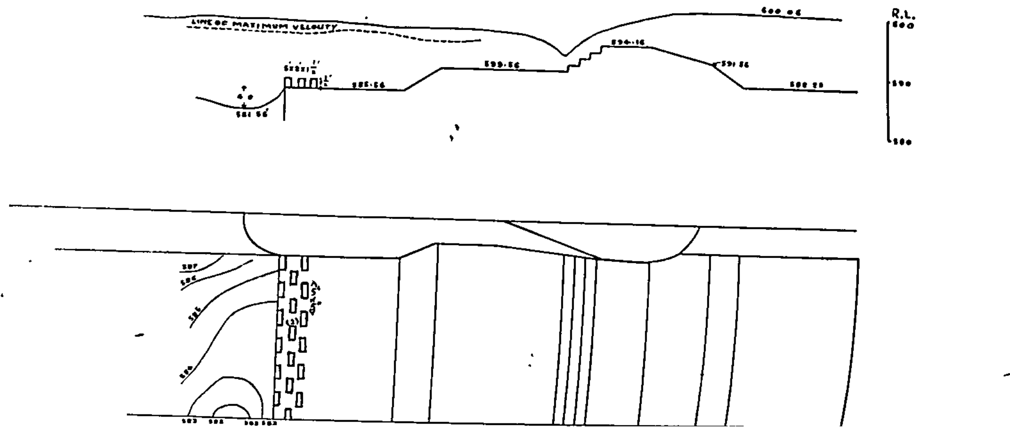


FIG. 139

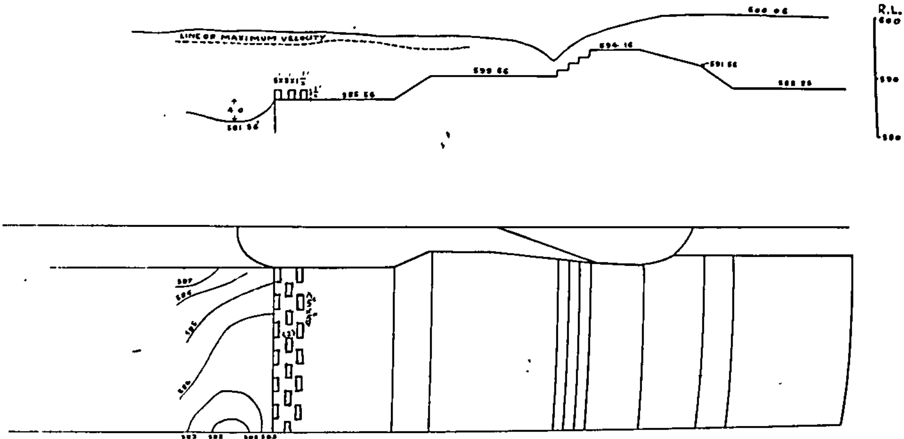
DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL

FULL SUPPLY

TWO FEET RETROGRESSION

WITH STAGGERED BLOCKS AT THE END OF THE FLOOR



DIPALPUR CANAL FALL AT R.D. 2,17,000

PARTIAL MODEL
FULL SUPPLY

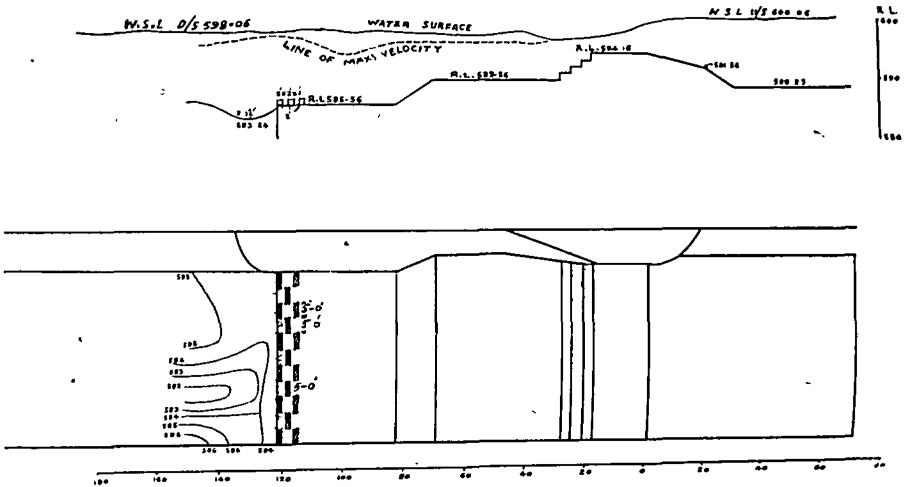


FIG.143

FALL AT R.D. 12500 MAIN BRANCH UPPER
SHOWING WAVE LAP BEFORE & AFTER RAISING CREST
MAXIMUM DIFFERENCE = 7·5 UNITS OF ORDINATE
" " = 8·5

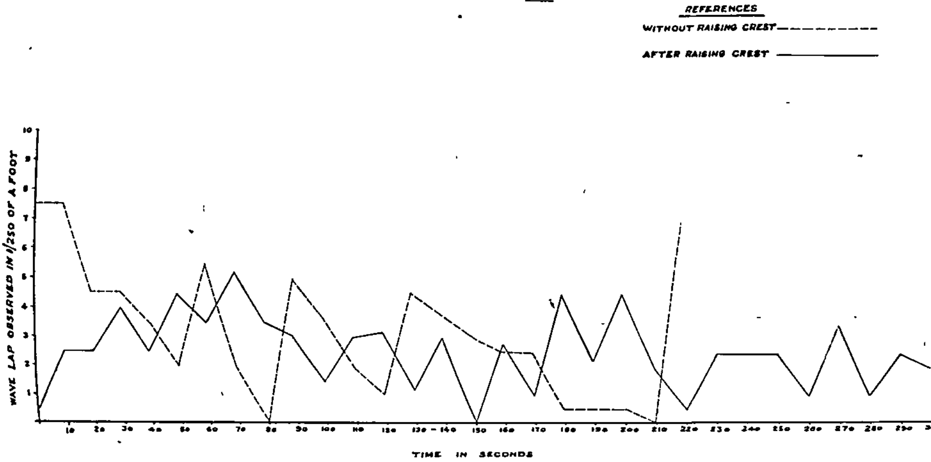


FIG. 144

RALIALI RAPID AT R. D. 11,900 MAIN BRANCH LOWER, UPPER BARI DOAB CANAL

Photograph showing the conditions of flow with the existing design

FIG. 145

RALIALI RAPID AT R. D. 11,900 MAIN BRANCH LOWER, UPPER BARI DOAB CANAL

Photograph showing the bed scour with the existing design

The model was first run for full supply conditions and observations for scour, water-surface profile, position of standing wave and wave lap were made. A plot of the observations is given in Figs. 142-143. A poor form of standing wave occurred on the glacis. Wave lap was observed at the downstream end of the work and swirls formed on both sides starting from the downstream end of the pier nose.

The crest of the rapid was next raised to R. L. 795·66 as proposed by the Superintending Engineer, Upper Bari Doab Canal Circle. The model was run with the discharge used before. The water level upstream obtained in this case was R. L. 800·26. An examination of the model showed that a well-defined standing wave formed on the sloping glacis. The wave lap was considerably diminished and the swirls were absent. The wave lap is plotted in Fig. 143. After closing down the model an inspection of the bed was made. It showed that the bed downstream was uniformly silted. There was no scour hole at any point on the bed. Side erosion was also very little. It may be concluded from this that the raising of the crest will not produce greater action at the bed and no extra protection is required against the bed erosion. Further experiments to reduce the wave lap are being devised.

Summary and Conclusion—A model of rapid at R. D. 12,500 Lahore Branch has been examined for the full supply discharge under the existing conditions and after raising the crest of the rapid. It has been shown that raising of the crest does not produce any increased action on the bed. There is, therefore, no necessity to construct any additional devices for the protection of the bed.

IV—*An investigation of a model of Ralhali Rapid at R. D. 11,900 Main Branch Lower, Upper Bari Doab Canal*

The Chief Engineer, in his letter No. 12960-S/101/1910, dated 24th October 1941, ordered an investigation to be made of a model of the rapid to determine methods to protect the bed and the sides against erosion taking place downstream of the work.

A model to horizontal and vertical scales of 1/7th was constructed at Malikpur. The channel for a length of 150 feet upstream and downstream of the rapid was also represented on the model. The bed of the channel above and below the rapid on the model constituted of pure sand of ·3 mm. diameter. The sides however were made with a mixture of sand and clay. A length equivalent to 50 feet on the model downstream of the work was pitched on both sides. A complete view of the model is given in Fig. 144.

Running the model under the existing conditions of the work—The model was first run with a discharge equivalent to the full supply in the channel, i. e., 2,700 cusecs, and with levels R. L. 797·91 upstream and 797·5 downstream. The conditions of flow obtained on the model with the above discharge are illustrated in Fig. 145.

The important indications obtained were :—

(1) A standing wave formed at a distance of 44·7 feet from the upstream end of the crest. The wave did not form uniformly over the entire width of the rapid. There was a well defined standing wave on the right side while on the left side for a length of 20 feet from the end there was only a forward moving jet and no proper standing wave was obtained. In this portion the forward moving jet was sometimes washed away and sometimes reformed.

(2) Downstream of the pacca bed due to the washing away of the forward moving jet shooting action occurred from the left towards the right side with a large back roller on the left side. This is illustrated in Fig. 145.

(3) A scour occurred on the right side and a shoal formed on the left. The scour observations are given in Fig. 146.

The experiments were repeated and similar results were again obtained. The indications obtained in the first experiment were confirmed.

The model was inspected at this stage by Khan Bahadur Mian Fakhr-ud-Din, Superintending Engineer, Upper Bari Doab Canal, who after inspection remarked that at the site too the jet was concentrated on the right. The model, therefore, indicated true conditions.

Raising the crest of the rapid—In the next experiment the crest of the rapid was raised by 2 feet as proposed by the Superintending Engineer, Upper Bari Doab Canal. The model with the raised crest was run with the conditions of flow tested before. The following points were noted :—

(1) The water level upstream of the crest as recorded on the model was R. L. 799·90.

(2) The standing wave formed at a distance of 42·75 feet from the upstream end of the crest. The wave formed uniformly along the entire width of the rapid.

(3) The jetting action downstream of the work was much less in this case than in the previous case. The jet was, however, unstable.

(4) The scour in the bed was uniform and was slightly less than that obtained before the crest was raised.

Tests with Staggered Blocks

(*i*) *Blocks at the toe of the glacis*—In order to reduce the action downstream it was decided to examine the effect of the construction of the staggered blocks. Two rows of Blocks, 5′ ×2′ and 1′ high, were constructed at the toe of the glacis and the model was run with full supply conditions for a period of 4 hours. At the end of the run the bed was surveyed. Plotting of the scour is given in Fig. 147.

FIG.146

FALL AT R.D. 11900 M.B.L.

SCALE {HOR=1/400, VER=1/100}

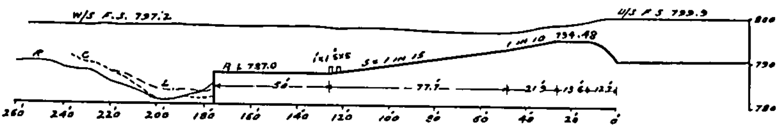


FIG 148

FALL AT R.D. 11900 M.B.L.

SCALE HOR = 1/400, VER = 1/200

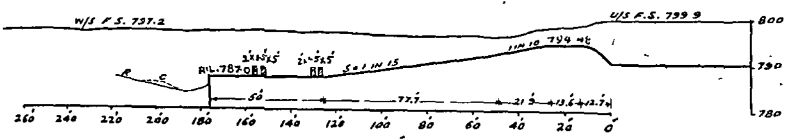


Fig. 149

RALIALI RAPID AT R. D. 11,900 MAIN BRANCH LOWER, UPPER BARI DOAB CANAL

FIG. 150

V.R. BRIDGE AT R.D. 40510 L.J. CANAL (MAIN LINE)

DISCHARGE PER FT. RUN 28·6 CUSECS

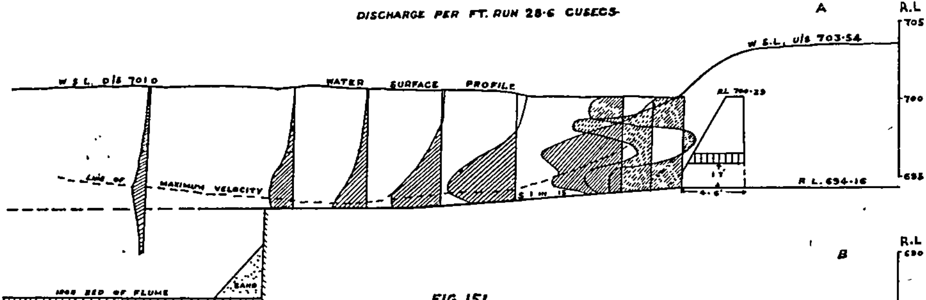
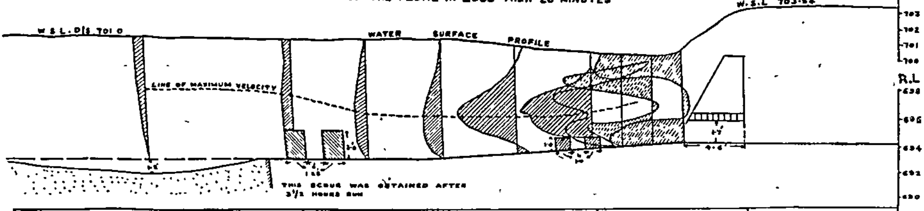


FIG. 151

THE WHOLE OF THE BED WAS SCOURED TO THE IRONPLATE OF THE FLUME IN LESS THEN 20 MINUTES



In the next experiment the height of the blocks was increased to 2 feet and the model was again run. Observations showed that 2 feet blocks at this position gave less scour and did not produce any surface *ripples.* The two feet blocks were, therefore, *superior to* 1 foot and in all further tests this height was used.

(ii) *Blocks downstream of the glacis*—As an alternative position, the blocks, 5′×2′ × 2′, were constructed in two rows on the horizontal pitching just downstream of the toe of the glacis and the model was run as before. A plot of the scour observation for this arrangement is given in Fig. 148. It was shown that the blocks at the toe of the glacis as used in (*i*) gave better results than this arrangement.

(iii) *Blocks on the horizontal pitching, 30 feet downstream of the end of the glacis*—The blocks from the downstream of the glacis were removed and fixed at 30 feet downstream of the end of the glacis. The portion between the blocks and the end of the glacis was pitched. The model was run as before and all the necessary observations were taken. It will be seen from an examination of Fig. 149, that this arrangement gives greater scour.

(iv) Next the blocks were constructed at 2 places, first just downstream of the end of the glacis and the second at a point 30 feet downstream of the end of the glacis. The model was run as before and all the necessary observations were taken. It was shown that this arrangement gave the most satisfactory results.

Recommendations—Two rows of blocks, 5′ × 2′ × 2′, should be constructed—

(*i*) just downstream of the end of the glacis.

(*ii*) 30 feet downstream of the end of the glacis.

The bed between the 2 lines of blocks should be grouted with cement.

V—*Investigation of a model of V. R. Bridge at R. D. 40,510, Lower Jhelum Canal for testing the Design for Reconditioning the work.*

The Chief Engineer in his U. O. No. 2562-N, dated the 18th November 1941, ordered an investigation of a model of the proposed design to raise the full supply level upstream of the bridge. It was desired by the Chief Engineer that the tests be completed by the 1st December 1941.

A sectional model of the design shown in *Fig.* 149a was constructed to the horizontal and vertical scales of 1/7 and examined. Coarse sand was placed upstream and downstream of the model to represent the canal bed. Two tests were made, one with the plain floor and the other with the staggered blocks in position. Observations of the water surface profile, line of maximum velocity and the depth of scour downstream were made. A plot of the observations made is given in Fig. 150.

Tests with the plain floor

When the model was run with the plain floor, very heavy action took place at the downstream end of the work. The sand at the downstream end which was level with the downstream floor at the commencement of the run was washed away in less than 15 minutes and the iron bed of the flume was exposed. This is shown in Figs. 150 and 151.

Tests with the staggered blocks in position

Staggered blocks were constructed at positions shown in Fig. 152. Sand was again placed in the flume downstream of the work and the model was run with the discharge used in the previous case for a period of $3\frac{1}{2}$ hours. A plot of the observations is given in Fig. 152 From an examination of this figure it will be seen that the line of maximum velocity is situated well above the floor level. A scour equivalent to 1·2′ occurred after $3\frac{1}{2}$ hours' run. The deepest point of the scour is situated away from the downstream end. The conditions of flow are illustrated in Fig, 153. The scour in this case is negligible as compared with that obtained in the previous case.

Conclusion—The design of the proposed wall without the staggered blocks is not satisfactory from the point of view of action downstream. However, when staggered blocks are added the design is improved considerably.

The model in operation with and without the staggered blocks was shown to the Under-Secretary, North, on the 1st of December 1941.

The Chief Engineer inspected the model in operation on the 4th December 1941 and altered the original design. The new design consisted of the following :—

(1) The breast wall to be constructed at R. L. 699·3 and no opening to be kept in this wall at the bottom.

(2) A baffle wall with top R. L. 695·4 to be constructed about 15 feet downstream of the breast wall.

(3) The floor downstream of the breast wall up to the baffle wall to be raised to R. L. 693·4.

The Chief Engineer ordered that the model should be re-examined after carrying out the above alterations in the design. After these alterations were carried out the model was re-examined. The observations for water surface profile, line of maximum velocity and the depth of scour were made. A plot of these observations is given in Fig. 154. From an examination of this figure it will be seen that the line of maximum velocity is deflected towards the top at the baffle wall. The water surface downstream of the baffle wall was very wavy. A scour equivalent to 3·8 feet deep occurred downstream of the work.

Fig. 152

V. R. BRIDGE AT R. D. 40,510 (LOWER JHELUM CANAL)

Plain Floor

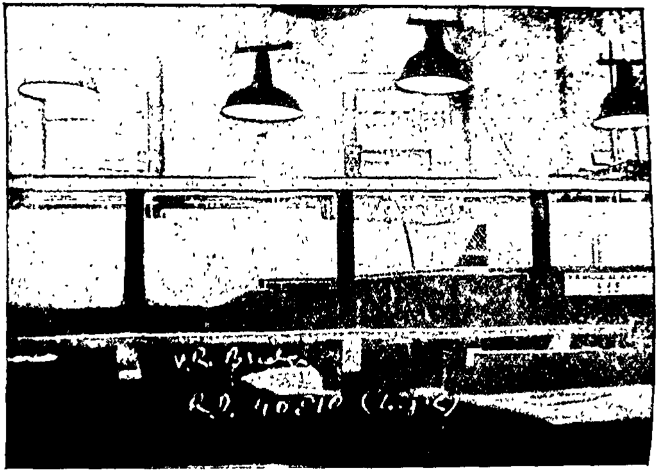
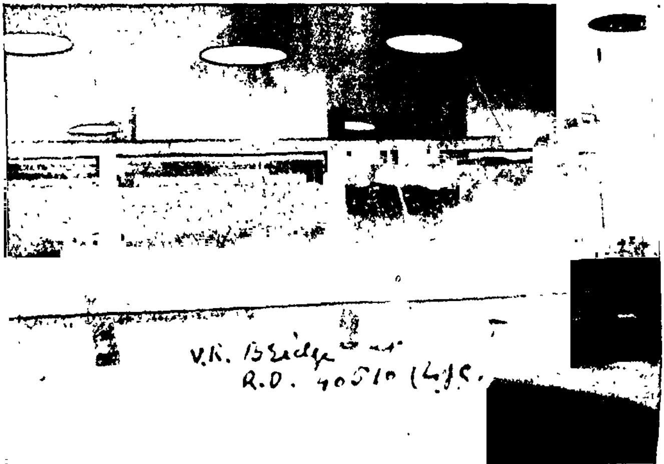


Fig. 153

Floor with Blocks



V.K. Bridge at
R.D. 40510 (L/S.

FIG. 154

V.R. BRIDGE AT R.D. 40510 L.J.C. (MAIN LINE)

SHOWING THE BAFFLE WALL AS PROPOSED BY C E.

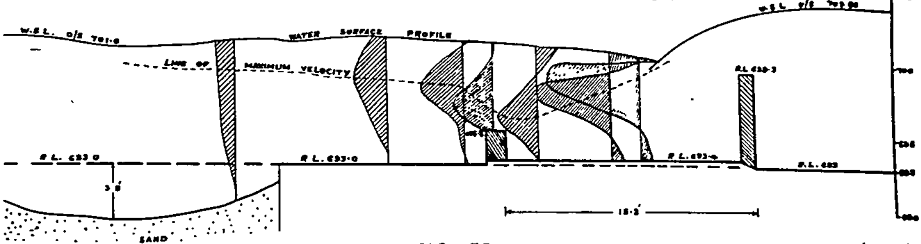


FIG. 155

THE BAFFLE WALL WAS SHIFTED DOWNSTREAM

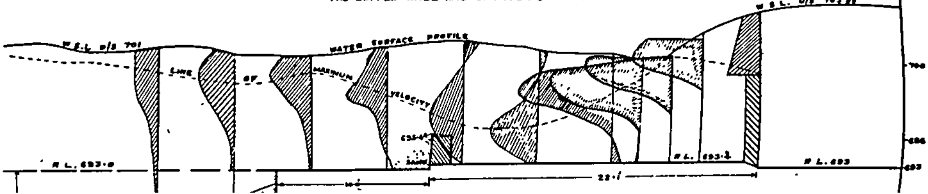


FIG. 156

V. R BRIDGE AT R. D. 40510 L.J. CANAL (MAINLINE)

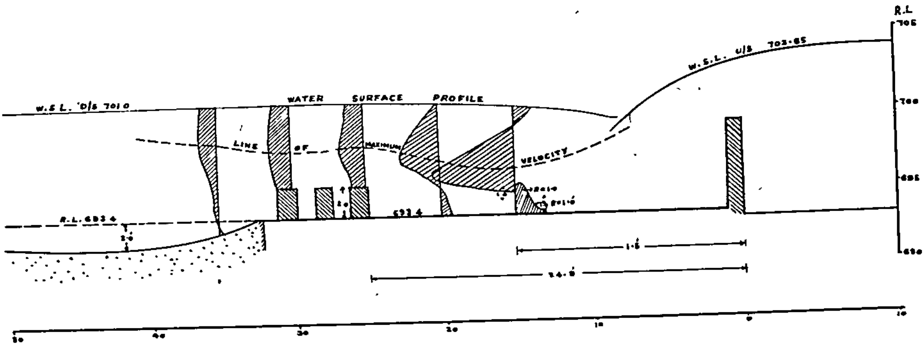
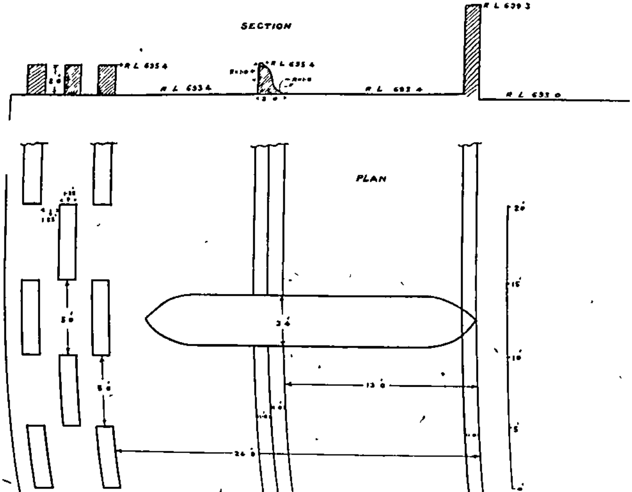


FIG. 157

V R BRIDGE AT R.D 40510 L J C. (MAIN LINE)



In the next test the baffle wall was shifted downstream in order to convert a greater length of the floor into a cistern (between the breast wall and the baffle wall). The results of the observations for this test are shown in Fig. 155. It will be seen from this figure that the shifting of the baffle wall away from the breast wall resulted in a greater scour downstream of the work. The baffle wall, therefore, was restored at its original position.

The Under-Secretary, North, informed on the 9th December 1941, that the floor downstream of the baffle wall was also to be raised to R. L. 693·4. This was done on the model and in addition, three rows of staggered blocks were constructed at the downstream end of the downstream floor. Running of the model showed that a baffle wall in conjunction with the blocks gave the most satisfactory results. The depth of scour was reduced considerably. However, the water surface downstream of the baffle wall was still wavy as the baffle wall being rectangular gave an abrupt kick. The design of the baffle wall was slightly changed and a double curve with a radius equivalent to 1 foot, was fitted upstream of the baffle wall. The results obtained are given in Fig. 156. This was the set which gave the most satisfactory results. The arrangement of the blocks in section and in plan is shown in Fig. 157.

C—Silt Experiments

The Chief Engineer, in his letter No. 9011 C. N./T/401/40, dated the 8th May 1941, ordered that experiments should be carried out in the laboratory flume with different grades of silt for obtaining suitable design for the silt extractors. A note dated 4th March 1941 by Mr. F. F. Haigh, the then Superintending Engineer, Upper Jhelum Canal Circle, on the above subject was also received from the Chief Engineer.

The experiments were, therefore, started according to the requirements laid down in Mr. Haigh's note.

The experiments were carried out in the 2·5 feet wide glass flume. A length of 20 feet of the flume was used for the model.

A sharp-crested weir was fixed at the upstream end of the model to produce a standing wave. The weir was made moveable so that it could be fixed at any desired position.

The escapage discharge could be measured in the calibration tank as well as at th[illegible] upstream end of th[illegible] fitte[illegible]ge tunnel was [illegible] over the silt.

For dropping silt into the water an endless belt working on two rollers across the whole width of the flume was constructed. Silt was spread over the cloth and dropped into the standing wave.

I—Experiments with a smooth bed representing a lined channel

In order to obtain conditions representing a lined channel the bed was made of seasoned teakwood. The surface was painted with

A perusal of the above table shows that results with 10 per cent extractor ratio require re-examination. There are several sources of error for instance water becoming dirty due to the working of other flumes and fungus growth on sand due to long running. The errors have been completely eliminated by taking necessary precautions.

D—Seepage and Regeneration Experiments

Attempts were made to study by means of models the forms of seepage flow from a canal and their effects on river supplies. Two methods were employed for this investigation :—

(1) The capillary tank method ; and

(2) the sand tank method.

In the capillary tank method, the flow of water took place between two glass plates separated by a distance of ·2 mm. The model was made in the back plate. The lines of flow were traced by introducing copying pencil points into the water at selected places. By this method clear and well-defined lines were obtained.

The sand tank method consisted of tracing the flow lines in sand by using potassium chromate and silver nitrate, the model being made in white sand. By this method permanent lines of flow could be obtained. The following different studies were made :—

(1) Seepage from a canal above a horizontal water-table.

(2) Regeneration of water in a canal from a horizontal water-table.

(3) Regeneration of water in a canal from a horizontal water-table when clay stratum occurs in the sandy sub-soil.

(4) Seepage from a canal above an oblique water-table.

(5) Regeneration of water in a canal from an oblique water-table.

(6) Regeneration of water in a river from the sub-soil water-table alone.

(7) Regeneration of water in a river from a canal situated at a higher level than the river bed.

Brief notes on each study are given below :—

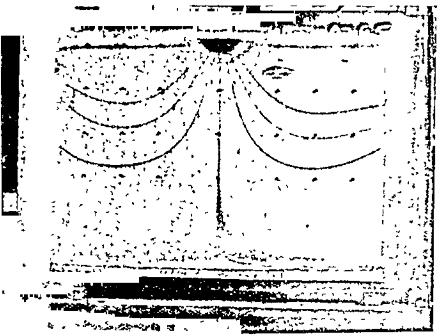
Study No. 1—Seepage from a canal above a horizontal water-table—This study was a simple case of flow from the canal into the subsoil. The subsoil water-table under the canal was maintained horizontal. The lines of flow obtained under these conditions of the experiment are illustrated in Fig. 158. It will be seen from an examination of this figure that the lines of flow are symmetrical. The central line is at right angles to the line of the subsoil water-table. Close to the bottom of the tank this line becomes diffused. In Fig. 159, the lines of flow as obtained by the sand tank method are shown. The lines of flow obtained in this case are exactly similar to those obtained by the capillary tank method.

Fig. 158

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 1

Seepage from the canal into a horizontal water-table



Lines of flow as obtained in the capillary tank

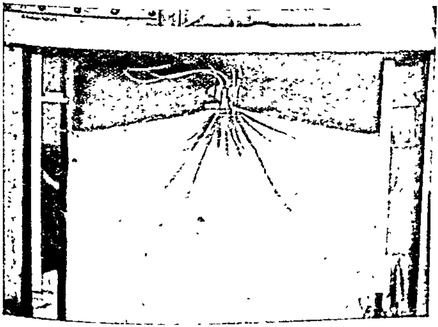
Water-level in the left chamber	..	87·420
Water-level in the right chamber	..	87·420
Water-level in the canal	..	90·735
Discharge	..	13·55 cc. per second

Fig. 159

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 1 (contd.)

Seepage from the canal into a horizontal water-table



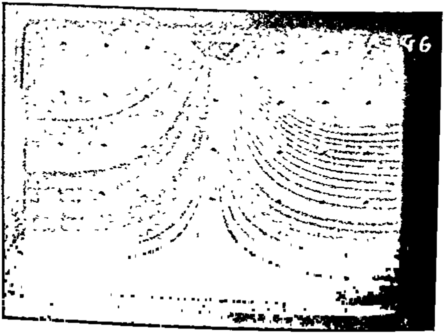
Lines of flow was obtained in a sand tank

FIG. 160

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 2

Regeneration into the canal from a horizontal water-table



Lines of flow as obtained in a capillary tank.

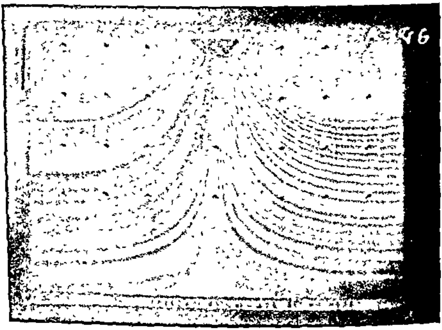
Water-level in the right chamber		90·700
Water-level in the left Chamber	..	90·675
Water-level in the canal	..	86·620
Discharge	..	12·87 cc. per second

FIG. 160

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 2

Regeneration into the canal from a horizontal water-table



Lines of flow as obtained in a capillary tank.

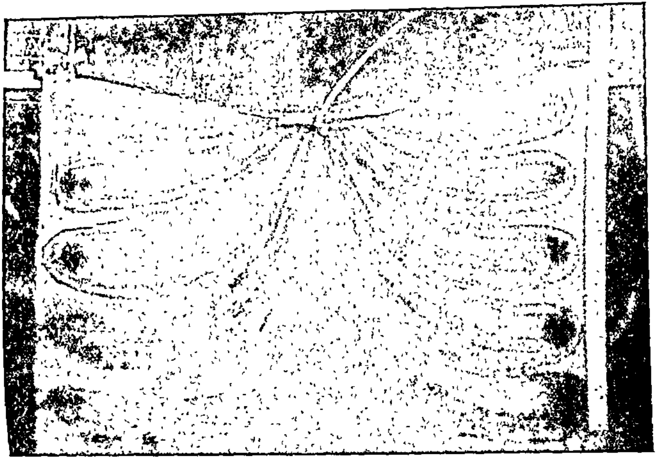
Water-level in the right chamber	90·700
Water-level in the left Chamber ..	90·675
Water-level in the canal ..	86·620
Discharge	12·87 cc. per second

FIG. 161

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 2 (contd.)

Regeneration into the canal from a horizontal water-table



Lines of flow as obtained in a sand tank

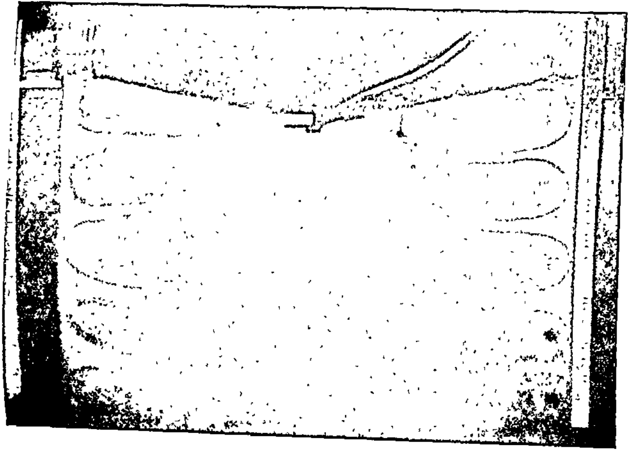
FIG. 161-A

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 2 (contd.)

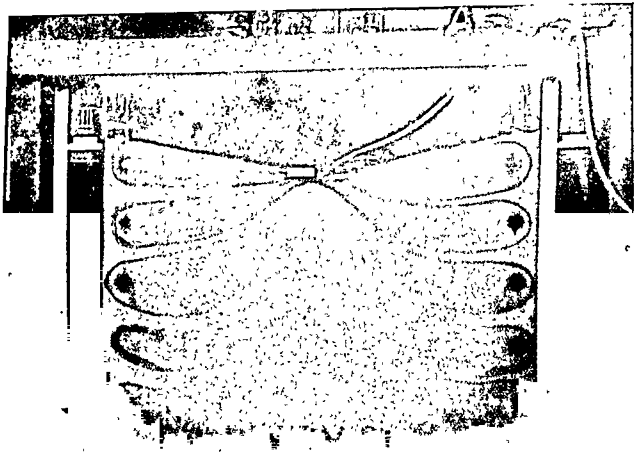
Regeneration into the canal from a horizontal water-table

Different stages of the development of the experiment



After one hour

Fig. 161-B

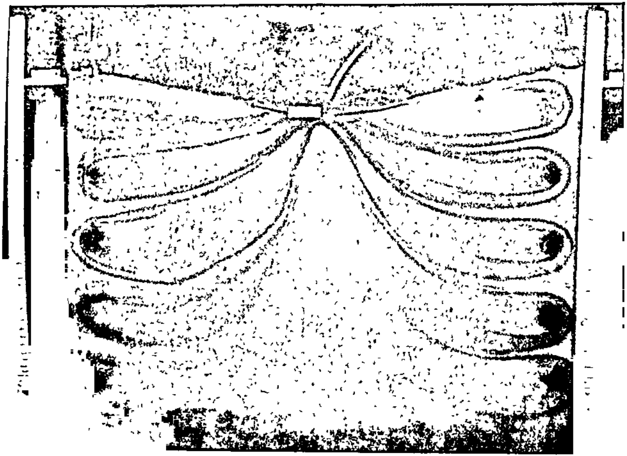


After two hours
Lines of flow as obtained in a sand tank

SEEPAGE AND REGENERATION EXPERIMENT

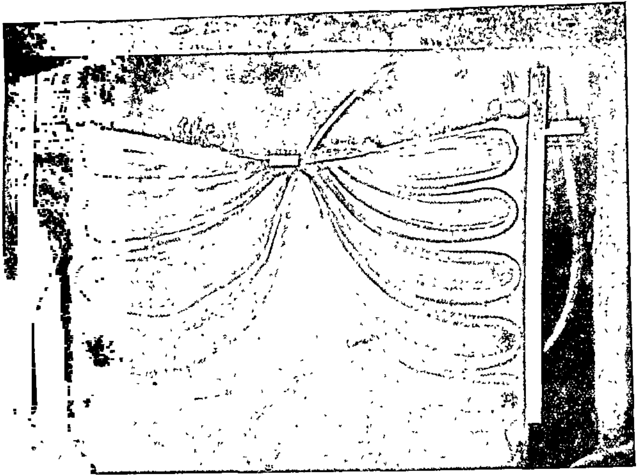
Study No. 2 (contd.)

Regeneration into the canal from a horizontal water-table



After 4 hours

FIG. 161-D



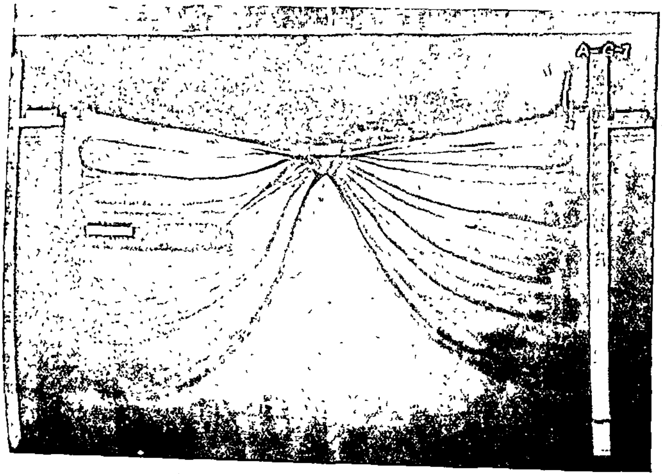
After 6 hours
Lines of flow as obtained in a sand tank

FIG. 162

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 3

Regeneration into the canal from a horizontal water-table in the presence of a clay strata in the sandy sub-soil



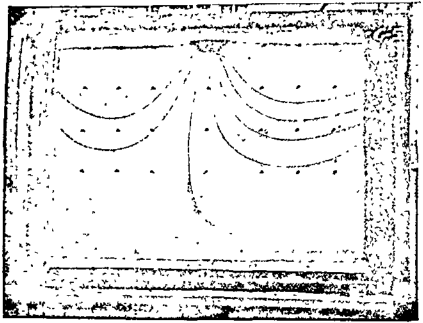
Lines of flow as obtained in a sand tank

FIG. 163

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 4

Seepage from the canal into an oblique water-table



Lines of flow as obtained inthe capillary tank.

Water-level in the right chamber	..	87·525
Water-level in the left chamber	..	87·430
Water-level in the canal	..	90·735
Discharge	..	13·60 cc. per second

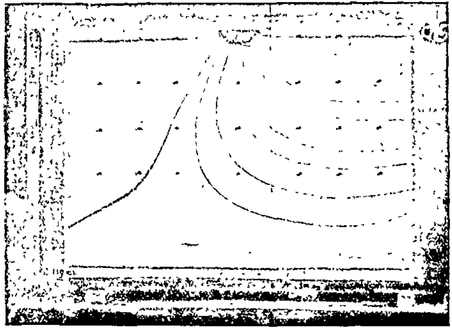
FIG. 164

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 4 (contd.)

Seepage from the canal into an oblique water-table

Slope on the water-table is steeper than in experiment No. 1



Lines of flow as obtained in a capillary tank.

Water-level in the left chamber	..	87·110
Water-level in the right chamber		83·860
Water-level in the canal	..	90·200
Discharge	..	13·45 cc. per second

Study No. 2—Regeneration of water in a canal from a horizontal water-table—The conditions of flow mentioned in Study No. 1 were reversed in this case, i.e., the canal level in this study was the same as the subsoil water-level in the former study, and the subsoil water-level was the same as the canal level in the previous case. The lines of flow by the capillary tank method are shown in Fig. 160. Fig. 161 shows the flow lines obtained in sand. In Figs. 161-A to 161-D the various stages of development of the lines of flow are illustrated. From an examination of these figures it will be seen that the lines of flow obtained in this study are exactly similar to those obtained in Study No. 1, excepting that the lines now obtained are in the reverse direction. The direction is indicated by the arrows marked on the photographs. The important indications obtained from this study are :—

(a) regeneration of water in the canal can take place from a considerable depth in the subsoil ;

(b) the velocity of flow taking place from deeper strata is lower than that from strata near ground surface.

(c) the forms of seepage flow and regeneration are completely reversible.

Study No. 3—Regeneration of water in a canal from a horizontal water-table when clay stratum occurs in the sand subsoil—The effect of the presence of a clay stratum in the otherwise sandy subsoil on the lines of flow was determined in this study. In the model of Study No. 2, a clay layer was introduced at the position shown in Fig. 162. The lines of flow were traced as before and are illustrated in the above figure. From an examination of this figure it will be seen that—

(a) the clay stratum in this position produces only a local effect. The lines of flow close to the clay layer are distorted while the flow lines farther away are not affected ;

(b) the presence of the clay strata causes an increase in the velocity of flow close to it.

Study No. 4—Seepage from a canal above an oblique water-table—In this study the effect of an oblique water-table (Instead of the horizontal) on the lines of seepage flow from the canal was determined. Three experiments were carried out in this case. In the first experiment the slope in the water-table was very small. The result of this is shown in Fig. 163. It will be seen from this figure that with a slight tilt in the water-table, the lines of flow from a 5/7th portion of the canal go towards the lower side.

In the second experiment the slope in the water-table was slightly increased. The effect is shown in Fig. 164. It will be seen from this figure that the flow from 6/7th portion of the canal moves towards the lower side.

In the third experiment the slope was still further increased and the result of this is shown in Fig. 165. The whole of the seepage flow from the canal went towards the lower side of the water-table.

An important indication obtained from this study was that the seepage from the canal is very sensitive to the obliquity of the water-table.

In the last experiment the seepage flow into the oblique water-table was traced. But the lines of flow in the water-table were not shown. This was now done. In order to obtain a clear distinction between the two sources of water, the lines of flow in the subsoil and the seepage lines were traced with different colours. Fig. 166 illustrates the lines of flow obtained. The indications obtained from this study were :—

(a) the lines of subsoil flow were depressed considerably by the seepage lines from the canal;

(b) the junction of the seepage and the subsoil flow is very sharp. No area of dead water or diffused flow exists at the junction.

Study No. 5—Regeneration of water in a canal from an oblique water-table—The conditions of flow of the last study were reversed in this case. The lines of flow obtained in this case are illustrated in Figs. 167 and 167-A. It will be seen from this figure that the flow in this case is exactly the reverse of that obtained in Fig. 166.

Study No. 6—Regeneration of a river from the subsoil water-table—In this study, the effects of subsoil flow on river regeneration were examined. The model, with the flow lines traced, is shown in Fig. 168. It will be seen from this figure that a portion of the subsoil flow enters the river bed.

Study No. 7—Regeneration of water in a river from a canal situated at a higher level than the river bed—In this study a model of a canal, a river and an oblique water-table, was examined. This is the most complicated case of ground water flow. The canal was situated at a higher level than the river and the sub-soil water. The slope in the water-table was maintained towards the river. The lines of flow in the subsoil as well as the seepage lines from the canal were traced and are shown in Fig. 169. The important indications obtained from this study are :—

(a) the subsoil flow lines are depressed considerably below the canal;

(b) almost the whole of the seepage from the canal appears as regeneration in the river.

E—Experiments on outlets

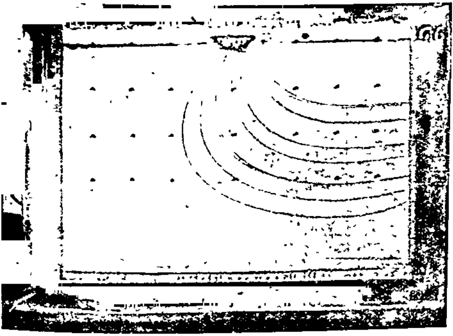
In order to examine in detail the different types of outlets in use in the Punjab and to evolve a most satisfactory type, a separate

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 4 (contd.)

Seepage from a canal into an oblique water-table

The Slope in the water-table is steeper than in experiment 2



Lines of flow as obtained in a capillary tank

Water-level in the left chamber ..	86·895
Water-level in the right chamber	81 995
Water-level in the canal ..	89·980
Discharge	14·40 cc. per second

FIG. 166

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 5

Seepage from the canal into an oblique water-table

Lines of flow in the water-table are also traced

FIG. 167

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 6

Regeneration into the canal from an oblique water-table

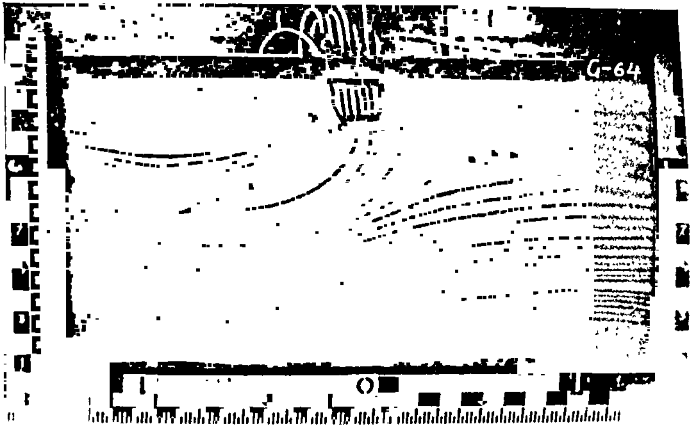
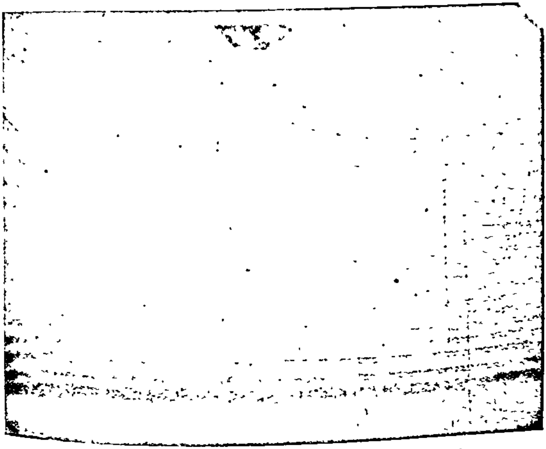


Fig. 167-A

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 6 (contd.)

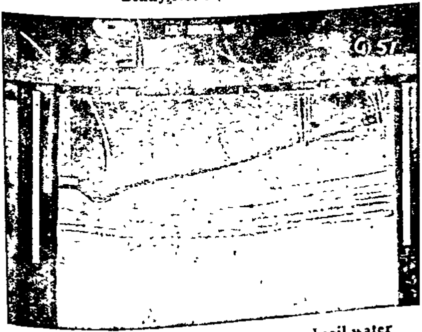


Regeneration into the canal from an oblique water-table

Fig. 168

SEEPAGE AND REGENERATION EXPERIMENTS

Study No. 6 (contd.)

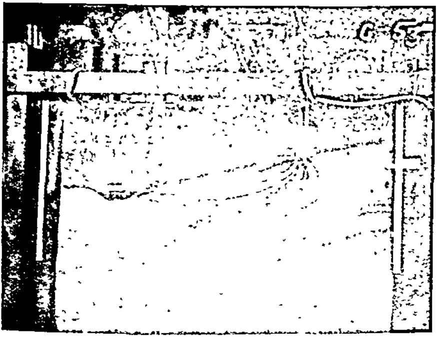


Regeneration of the river from the subsoil water

Fig. 169

SEEPAG AND REGENERATION EXPERIMENTS

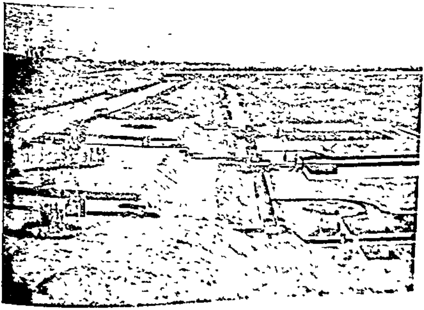
Study No. 7



Regeneration of the river from a canal situated close to the river at the higher level

Fig. 170

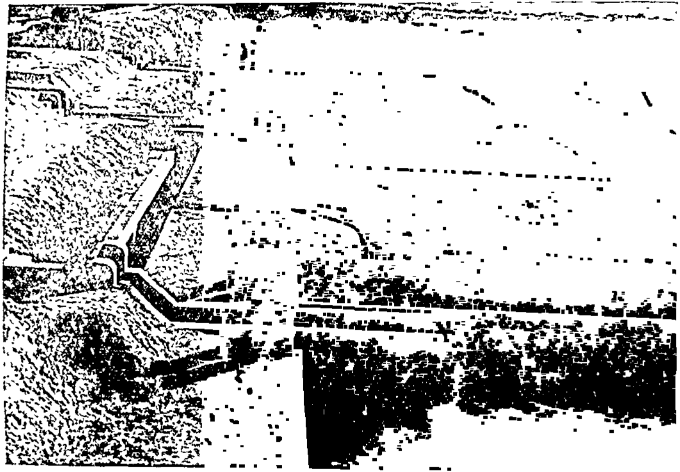
OUTLET EXPERIMENTS AT JOYANWALA



A general view of the station

Fig. 171

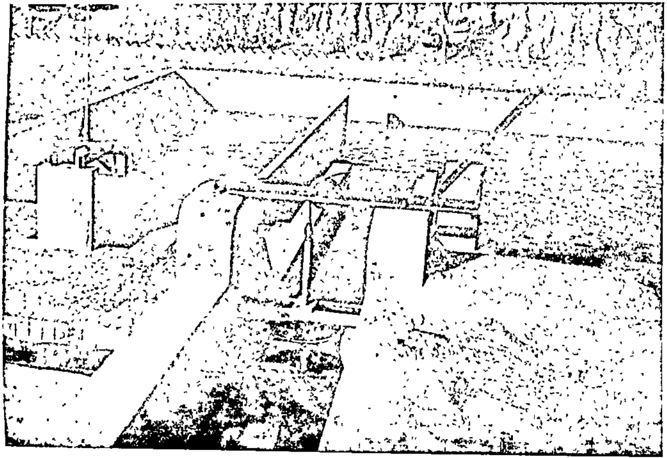
OUTLET EXPERIMENTS AT JOYANWALA



A closer view of the Experimental station

FIG. 172

OUTLET EXPERIMENTS AT JOYANWALA



A view of the diversion valve

Research Station was established last year at Joyanwala at R. D. 221,000 Main Line Lower, Upper Chenab Canal. At this place there is a ten feet fall in the canal and therefore plenty of head was available for the experiments. It was decided to construct an independent channel for the experiments. For this purpose an area of 6 acres was acquired on the right side of Chichoki-Malian Distributary and the following works were constructed :—

(1) Head Regulator.

(2) Crossing over Chichoki-Malian Distributary.

(3) Experimental Channel.

(4) The Calibration Tanks and the Escape Channel.

(5) Syphon under Chichoki-Malian Distributary and Outfall.

1. *Head Regulator*—The head regulator was constructed in the existing gauge well on the upstream of the fall on the right side. A gate was fitted to regulate the supply into the channel. A four feet diameter Hume-Pipe was used for the regulator. The maximum discharge capacity of the regulator was about 100 cusecs.

2. *Crossing over Chichoki-Malian Distributary*—As Chichoki-Malian Distributary was between the Head Regulator and the station area the experimental channel had to be carried over the distributary. For this purpose a Hume-Pipe crossing was constructed over the distributary. The pipes rested on iron trestles which in turn were supported on drum foundations.

3. *Experimental Channel*—The Experimental Channel is 1,200 feet in length. The bed width of the channel is 13·5′ and the full supply depth is 2·35 feet. The outlets were constructed in the central portion of the channel. At the end of the experimental channel a six feet fall was provided to discharge the water of the experimental channel into the escape channel.

4. *The Calibration Tanks and the Escape Channel*—Three calibration tanks, each 25′ × 25′ × 3′, were constructed on the right side of the experimental channel at a central distance of 50 feet. Each tank was provided with an inlet flume and a sluice valve. For the measurement of water level in the tanks a gauge well 3′ × 1·5 × 3′ fitted with a pointer gauge reading up to 1/1000th of a foot was constructed. In the inlet flume a diversion arrangement was fitted which connected the water-course to the tank or to the escape channel as desired. The water from the calibration tank discharged into the escape channel through the sluice valve. A view of the experimental channel, the escape channel, the calibration tanks and the diver[illegible] arrangement is shown in Figs. 170, 171 and 172.

5. *Syphon under Chichoki-Malian Distributary and Outfall*—The escape channel was carried under the Chichoki-Malian Distributary through a Hume-Pipe syphon. The water in the escape channel discharged into the canal and an outfall was constructed at that point.

The Programme of Work—The experiments were first carried out on Crump's Adjustable Proportionate Module for the co-efficient of discharge and the submergence limits. Before carrying out the actual tests it was decided to determine the minimum distances at which the presence of one outlet did not influence the other. For this test three similar outlets were constructed on the experimental channel. It was found that when the outlets were placed 50 feet apart, the presence of one outlet did not influence the other. In all further tests therefore three outlets were examined simultaneously. On each outlet a trolly fitted with straight edges as shown in Fig. 173 was constructed for making accurate measurements. A gauge well was also fitted about five feet upstream of each outlet for measuring the water-level in the experimental channel. The following A. P. Ms. were examined for the co-efficient of discharge and submergence limits :—

A. P. Ms. with B = ·2′, ·25′, ·32′, ·4′, ·5′, ·63′, ·80′ and 1′. Special forms were prepared for recording the daily observations taken on the outlets. A specimen is given below.

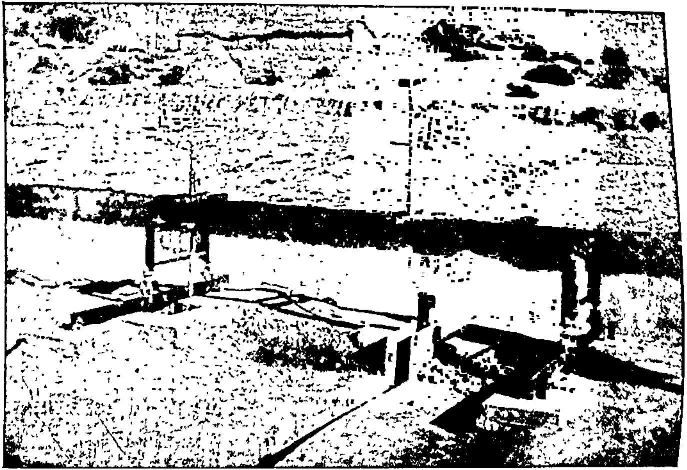
The data obtained were calculated and plotted. A sample of the results is given in Tables XIII and XIV, and also plotted in Figs. 174—79. A detailed report on the observations made for this portion is under preparation for submission to the Chief Engineer separately.

OUTLET EXPERIMENT (BLANK FORM)

Series——		B. M. Reading		=
Setting at——		Water Surface Level	I	=
Discharge of parent channel——		Ditto	II	=
Width of orifice	=	Ditto	III	=
Sill Reading	=	Ditto	IV	=
Top Reading	=	Mean		=
Height of orifice, Y	= +	Depth of water over Crest, H		=
		B. M. Reading		=
		Crest Reading		=
		Water Surface Level	I	=
		Ditto	II	=
		Ditto	III	=
		Ditto	IV	=
		Ditto	V	=
		Ditto	Critical	=
Date of observation		M. M. Working head, hm		=

FIG. 173

OUTLET EXPERIMENTS AT JOYANWALA



Outlet fitted with trolly arrangement

5. *Syphon under Chichoki-Malian Distributary and Outfall*.—The escape channel was carried under the Chichoki-Malian Distributary through a Hume-Pipe syphon. The water in the escape channel discharged into the canal and an outfall was constructed at that point.

The Programme of Work—The experiments were first carried out on Crump's Adjustable Proportionate Module for the co-efficient of discharge and the submergence limits: Before carrying out the actual tests it was decided to determine the minimum distances at which the presence of one outlet did not influence the other. For this test three similar outlets were constructed on the experimental channel. It was found that when the outlets were placed 50 feet apart, the presence of one outlet did not influence the other. In all further tests therefore three outlets were examined simultaneously. On each outlet a trolly fitted with straight edges as shown in Fig. 173 was constructed for making accurate measurements. A gauge well was also fitted about five feet upstream of each outlet for measuring the water-level in the experimental channel. The following A. P. Ms. were examined for the co-efficient of discharge and submergence limits :—

A. P. Ms. with B = ·2′, ·25′, ·32′, ·4′, ·5′, ·63′, ·80′ and 1′. Special forms were prepared for recording the daily observations taken on the outlets. A specimen is given below.

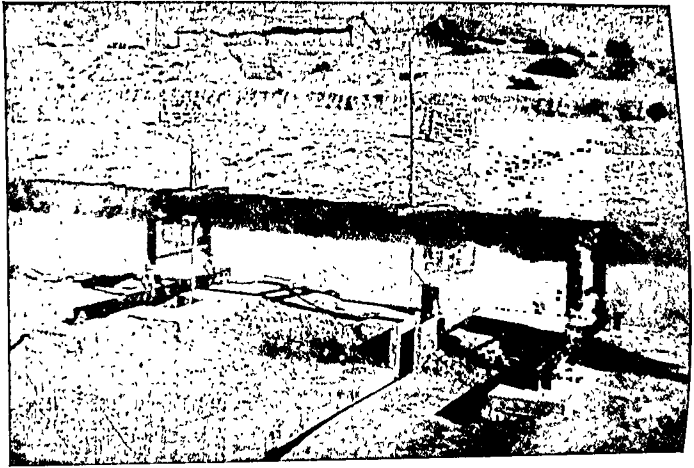
The data obtained were calculated and plotted. A sample of the results is given in Tables XIII and XIV, and also plotted in Figs. 174—79. A detailed report on the observations made for this portion is under preparation for submission to the Chief Engineer separately.

OUTLET EXPERIMENT (BLANK FORM)

Series————			B. M. Reading		=
Setting at————			Water Surface Level	I	=
Discharge of parent channel————			Ditto	II	=
Width of orifice	=		Ditto	III	=
Sill Reading	=		Ditto	IV	=
Top Reading	=		Mean		=
Height of orifice, Y	=	+	Depth of water over Crest, H		=
			B. M. Reading		=
			Crest Reading		=
			Water Surface Level	I	=
			Ditto	II	=
			Ditto	III	=
			Ditto	IV	=
			Ditto	V	=
			Ditto	Critical	=
Date of observation			M. M. Working head, hm		=

FIG. 173

OUTLET EXPERIMENTS AT JOYANWALA



Outlet fitted with trolly arrangement

FIG. 174

JOYANWALA EXPERIMENTAL STATION

DIAGRAMMATIC SKETCH OF DIVERSION VALVE

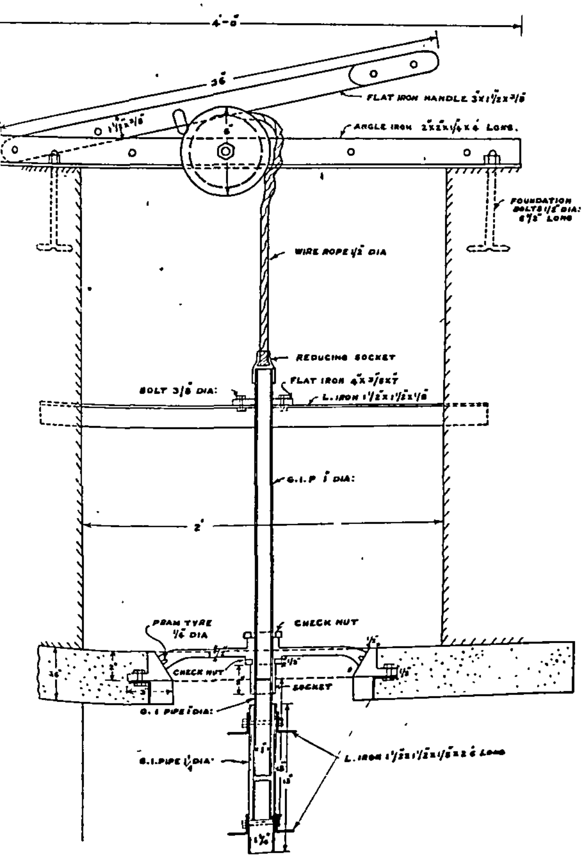
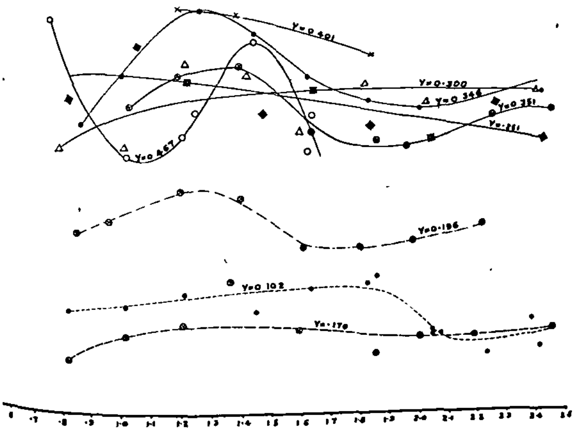


FIG. 175

RELATION BETWEEN $\ddot{H}$ AND $\ddot{C}$ FOR VARYING VALUES OF $\ddot{Y}$

B = 0·80

SETTING AT BED LEVEL

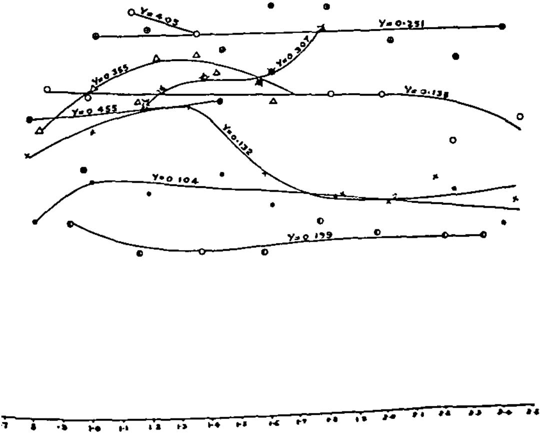


DEPTH OF WATER OVER CREST IN FEET $\ddot{H}$

FIG. 176

RELATION BETWEEN "H" AND "C" FOR VERYING VALUES "Y"

B = 1·00
SETTING AT BED LEVEL



DEPTH OF WATER OVER CREST IN FEET "H"

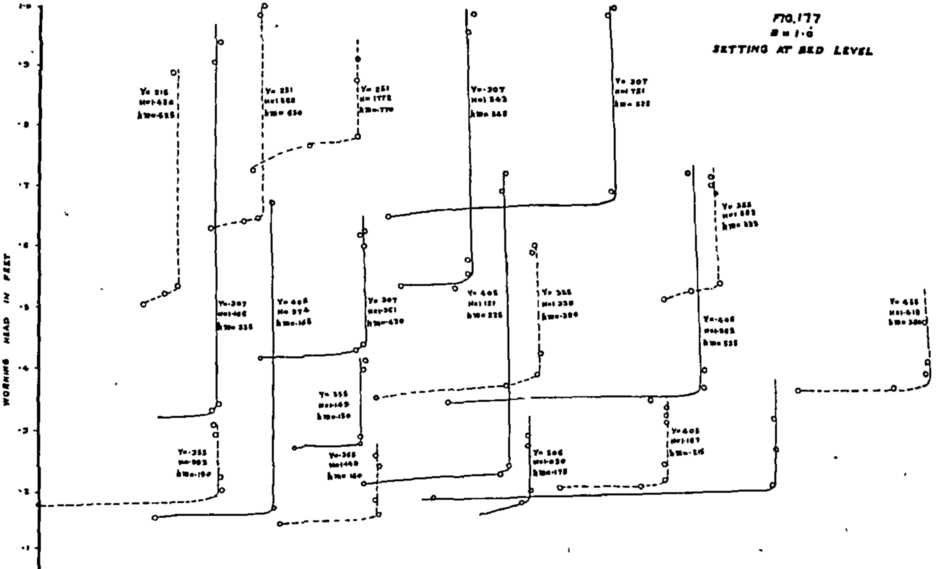


FIG. 177
B = 1·0
SETTING AT BED LEVEL

FIG. 178

RELATION BETWEEN H AND h m FOR VERYING VALUES OF Y

B = 0·8'

SETTING AT BED LEVEL

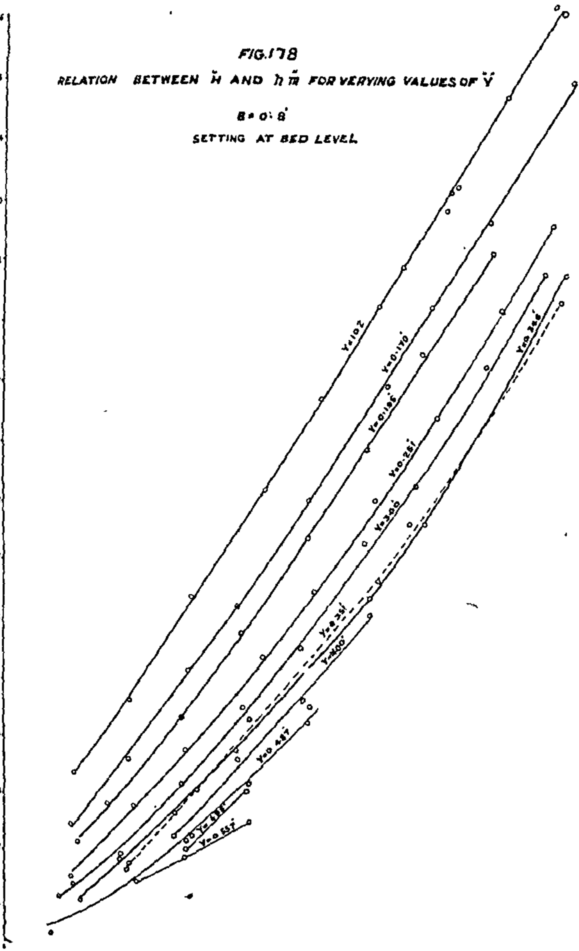
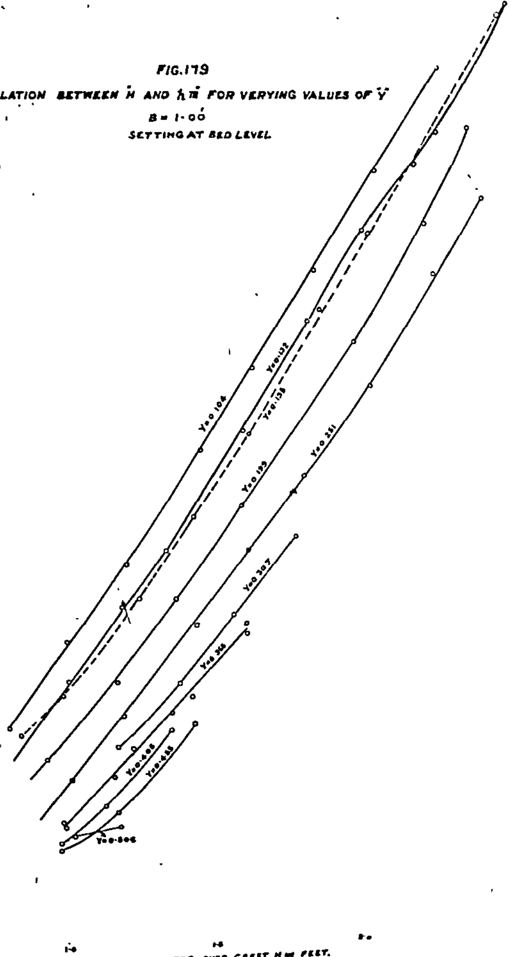


FIG. 179

RELATION BETWEEN H AND h m FOR VERYING VALUES OF Y

B = 1·00'

SETTING AT BED LEVEL



DEPTH OF WATER OVER CREST H IN FEET.

	I Observation	II Observation	III Observation	IV Observation with W.H.=hm	V Observation with W.H.=.02 +(hm)	VI Observation with W.H.= hm—·02
B. M. Reading =						
Crest reading =						
Water Surface level I =						
Ditto II =						
Ditto III =						
Ditto IV =						
Mean Surface level =						
Depth of water over Crest, H =						
Water surface level in tank Initial =						
Ditto Final =						
Difference =						
Volume of water V =						
Time in seconds, t =						
Discharge of outlet $q = \frac{V}{t}$ =						
Depression Head h=H—Y =						
Co-efficient $C = \frac{q}{BY\sqrt{h}}$ =						
B. M. Reading =						
Sill reading of sharp-edged weir =						
Water Surface level I =						
Ditto II =						
Ditto III =						
Ditto IV =						
Mean Surface level =						
Depth of water over Sill, hw =						

TABLE I

Model of Kalabagh Weir

COMPARISON OF WATER SURFACE LEVELS ON THE MODEL AND ON THE PROTOTYPE WITH A DISCHARGE OF 31,308 CUSECS IN THE RIVER.

Right Creek

Reduced distances along the bela with U/S end as 0.	Water surface levels observed on the prototype	Water surface levels observed on the model
	R. L.	R. L.
0	685·26	685·30
500	684·10	683·15
1,000	683·24	682·45
1,500	682·35	681·75
2,000	682·20	681·75
2,500	681·87	681·60
3,000	680·83	680·75
3,500	680·38	679·10
4,000	679·63	678·80
4,500	679·11	678·60
5,000	678·24	677·05
5,500	677·00	676·90
6,000	676·41	676·05
6,500	675·62	672·20
7,000	675·52	675·05
7,500	675·40	675·30
8,000	675·39	675·30
8,500	675·36	675·20
9,000	675·36	675·20
9,500	675·32	675·25
10,000	675·31	675·20
10,500	675·29	675·20
11,000	675·23	674·80
11,500	675·19	674·50
12,000	675·15	674·50
12,500	675·10	674·50
13,000	675·03	674·50
13,500	674·66	674·05
14,000	674·87	674·05
Tail	674·74	674·05

TABLE II

Model of Kalabagh Weir

COMPARISON OF WATER SURFACE LEVELS ON THE MODEL AND ON THE PROTOTYPE WITH A DISCHARGE OF 31,308 CUSECS IN THE RIVER.

Centre Cree:

Reduced distance along the bela with U/S end as 0	Water surface levels observed on the prototype	Water surface levels observed on the model
	R. L.	R. L.
0	685·26	685·60
500	85 06	85·00
1,000	85·06	84·90
1,500	84·94	84·10
2,000	84·34	..
2,500	84·09	84·10
3,000	81·67	83·80
3,500	79·60	83·50
4,000	78·76	82·90
4,500	78·50	82·15
5,000	78·49	79·75
5,500	78·45	79·00
6,000	78·26	78·20
6,500	77·31	77·50
7,000	77·04	77·35
7,500	76·95	77·35
8,000	76·75	77·35
8,500	76·55	76·70
9,000	76·39	76·35
9,500	76·39	76·05
10,000	76·27	75·95
10,500	76·03	75·95
11,000	75·22	75·85
11,500	75·79	75·85
12,000	75·62	75·55
12,500	75·24	75·85
13,000	75·20	74·50
13,500	74·74	74·50

TABLE

Experiments on

GAUGES AT DIFFERENT

Discharge

Time	Kalabagh Railway Bridge	Nawab Bund	U/S of First Diversion Bund	D/S of First Diversion Bund	U/S of 2nd Bund
1	2	3	4	5	6
	R. L.	R. L	R. L.	R. L.	R. L
12·20	686·5	656 4	686·43	685·56	684·15
12·45	86 6	86·5	86·53	85·36	84·07
1·00	86·6	86 6	86·53	85·67	84·29
1·15	86 6	86 5	86·53	85·67	84·29
1·30	86·6	86 6	86 53	85·67	84·15
1·45	86 6	86·6	86 64	85 56	84 29
2·0	86·6	86·6	86·53	85·67	84·29
2 15	86 6	86·58	86·53	85·56	84·29
2.30	86·8	86 8	86·76	84·61	84·54
2·45	86 65	86·6	86·68	84·39	84·39
3 0	86·63	86·6	86·68	84·29	84·39
3·15	86 8	86·5	86·74	84·29	84·5
3 30	86 8	86·7	86·74	84 29	84·61
3·45	86 8	86·7	86·74	84·29	84·61
4·0	86 8	86·7	86·74	84·29	84·61
4·15	87 2	87·2	87·28	86·58	85·25
4·30	87 2	87·2	87·28	86·86	85·25
4·45	87 2	87·2	87·28	86·86	85·25
5·00	87·2	87·2	87·28	86·86	85·25
5 15	87·6	87 7	87·61	87·39	85·68
5 30	87 6	87·7	87·61	87·39	85·57
5·45	87 6	87 7	87·61	87·39	85·57
6·00	87·6	87·7	87·61	87·28	85·57
6 15					
6 30	88 0	87·9	88·1	87·71	86·06
6·45	88 2	88·2	88·25	87·82	86·00
6·50	88·2	88 3	88·25	87·82	85·99
7·15	88·2	88·3	88·25	87·82	85·99
7·30	88 2	88·3	88·25	87·82	85·99
7 45	88·2	88·3	88·25	87·82	85·99
8 00	88 2	88 3	88·35	87·62	85·99
8·30	88·2	88·4	88·25	87·93	86·31
8·45	88·2	88 4	88·47	87·93	86·31
9 00	88·6	88 4	88·47	87·93	86·31
9 15	88 7	88·6	88 57	88·25	86·86
9 45	88 8	88 8	89·42	88·46	86·86
10·15	89 0	88·8	88 89	88·57	88·05
10 30	88·6	89 0	89·0	88·57	87·50
11·00	88 7	89·0	89·0	88·57	87·50
11·15	90·2	90·4	90·39	90·28	90·28
11·45	90·4	90 6	90·61	90·71	90·6
12·15	90·6	90 8	90·71	90·6	90·6

TABLE

Experiments on

GAUGES AT DIFFERENT

Discharge

Time	Kalabagh Railway Bridge	Nawab Bund	U/S of First Diversion Bund	D/S of First Diversion Bund	U/S of 2nd Bund
1	2	3	4	5	6
	R. L	R. L	R. L.	R. L.	R. L.
12 20	686 5	686 4	686·43	685·56	684·15
12·45	86 6	86 5	86·53	85·36	84·07
1·00	86 6	86 6	86·53	85·67	84·29
1 15	86 6	86 5	86·53	85 67	84·29
1·30	86·6	86 6	86·53	85·67	84·18
1·45	86·6	86 6	86·64	85·56	84·29
2·0	86·6	86 6	86·53	85·67	84·29
2 15	86 6	86 58	86·53	85·56	84·29
2.30	86 8	86·8	86·76	84·61	84·54
2·45	86 65	86·6	86·68	84·39	84·39
3 0	86·63	86·6	86·68	84·29	84·39
3·15	86 8	86 5	86·74	84·29	84·5
3 30	86 8	86 7	86·74	84·29	84·61
3 45	86 8	86·7	86·74	84·29	84·61
4·0	86 8	86·7	86·74	84·29	84·61
4 15	87 2	87·2	87·28	86·58	85·25
4·30	87·2	87·2	87·28	86·86	85·25
4·45	87 2	87·2	87·28	86·86	85 25
5 00	87·2	87·2	87·28	86·86	85·25
5·15	87 6	87·7	87·61	87·39	85·68
5 30	87·6	87 7	87·61	87·39	85·57
5·45	87 6	87·7	87·61	87·39	85·57
6 00	87 6	87·7	87·61	87·28	85 57
6 15					
6 30	88 0	87 9	88·1	87·71	86·06
6·45	88 2	88 2	88·25	87·82	86·00
6·50	88 2	88 3	88·25	87·82	85·99
7·15	88·2	88 3	88·25	87·82	85·99
7·30	88 2	88·3	88·25	87·82	85·99
7 45	88·2	88 3	88·25	87·82	85·99
8 00	88 2	88 3	88·35	87·62	85·99
8 30	88·2	88·4	88·25	87·93	86·31
8 45	88·2	88·4	88·47	87·93	86·31
9 00	88·6	88 4	88 47	87·93	86·31
9 15	88 7	88·6	88 57	88·25	86·86
9 45	88 8	88 6	89 42	88·10	86·86
10·15	89 0	88·8	88·89	88·57	86·05
10 30	88 6	89 0	89·0	88·57	87·50
11 00	88 7	89 0	89·0	88·57	87·50
11·15	90 2	90·4	90·30	90·28	90·28
11·45	90·4	90·6	90·61	90·71	90·0
12 15	90·6	90·8	90·71	90·6	90·0

III

River Diversion

PERIODS

—31,000 cusecs.

Intermediate No. 1	Intermediate No. 2	D S of 2nd Diversion Bund	U/S of 1st leading cut	U/S of 2nd leading cut	D/S leading cut
7	8	9	10	11	12
R. L.	R. L.	R. L.	R. L.	R. L.	R. L.
684·71	682·57	682·79	684·71	683·64	—
84·82	82·46	82·04	84.71	81·5	675·2
84·82	82·67	82·14	84·71	81·28	75·2
84·82	82·46	82·04	84·71	81·28	75·2
84·61	82·67	81·93	84·61	81·8	75·2
84·71	82·57	81·93	84·71	81·07	75·2
84·71	82·57	81·93	84·71	81·07	75·35
84·71	82·57	82 04	84·71	81·07	75·3
84·82	83·21	83 21	84·81	82·08	75·4
84·78	82·57	82·04	84·78	81·07	75·38
84·80	82·57	81·93	84·80	81·07	75·4
84·82	82·46	82 04	84·92	81·07	75·4
84·82	82·57	82·04	84 17	81·07	75·4
84·82	82 57	82 04	84·92	81·07	75·4
84·82	82 57	82 04	84·92	81·07	75·2
85 03	82 68	82·25	85 14	81·39	75·4
85·03	82·68	82·35	85·14	81·39	75·4
85·03	82 68	82·35	85·14	81·39	75·4
85·03	82 68	82·35	85·14	81·39	75·4
85·25	82·79	82 57	85·25	81·39	75·5
85·25	82·89	82·65	85·25	81·39	75·5
85·25	82·89	82·68	85·14	81·28	75·5
85 25	82·89	82·57	84·70	81·28	75·5
..	.	.	84 92	81·39	—
85·57	83 21	83·11	85·14	81·71	75·6
85·57	83 21	83·43	85 14	81 71	75·5
85 67	83·42	84·50	85 25	83·42	75·5
85·67	83·42	84·50	85·25	83·45	..
85·67	85·99	85 67	85·25	85·14	76·6
85·79	85·99	85 79	85 14	85 35	76·4
85·67	86 79	85 67	84 71	85·35	76 4
86·75	86 53	86·43	85 79	85 79	76 6
86 75	86·53	86 53	85·57	85·74	76·40
86·64	86·53	86 53	85 57	85·74	76·4
86 64	87·50	86 86	87 07	86·64	[illegible]
86 86	87·50	87·61	86 86	86·0	[illegible]
87·93	87 39	87·39	87 29	86·25	[illegible]
88·25	88 1	87·39	87 07	87·39	[illegible]
88 36	88·14	87·39	[illegible]	87 07	[illegible]
89·96	89·75	89·21	[illegible]	89 11	[illegible]
90·28	89·96	89·04	[illegible]	89 11	77 0
90·17	89·85	89·00	[illegible]	89 11	77 0

TABLE IV

Model of Kalabagh Weir

EXPERIMENTS ON RIVER DIVERSION

Time of observation	Ratio of discharge between the central and right creek		Remarks
	Right	Centre	
12·15 p. m	2·74	1	Before diversion.
12·45 "	2·22	1	Right creek partially closed.
2·0 "	1·88	1	Right creek further closed.
5·0 "	0·52	1	Ditto ditto.
5·45 "	0·35	1	Ditto ditto.
			Right creek completely closed at 6-15 p. m.

TABLE V

Conditions of experiment	Observations of the Left main cut			Left subsidiary cut			Right main cut			Right subsidiary cut			Downstream main cut		
	Velocity	Width	Depth	V	B	D	V	B	D	V	B	D	V	B	D
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Time 12 45 p. m.															
Cuts after running for 15 minutes	1·9	1 38	·14	.	.	.	1 30	1 17	·043	..	..	..	·64	1·67	·43
Time 2.30 p. m.															
Stones damped on rapid	1·8	1·4	·15	..	.	..	1·20	1·17	·043	..	..	..	·67	1·07	·48
Time 4.30 p. m.															
More stones damped	2·15	1 6	·18	2 0	·71	·13	1 72	1·30	·07	.	..	..	..	..	..
	2·14	2 25	·25	2 0	1·16	·20	2 00	1 31	·13	1·67	·83	·10	1·97	2·30	·44
Time 6.45 p. m.	2·0		25	2·0	..	·16	2·10	..	·20	1·74	..	..	1·53	..	..
Time 7.0 p. m.—															
Central creek partially closed.	2·0	2·17	·35	2·5	1·42	·17	2·67	1·67	·30	2·14	1·10	·25	4·0	2·44	·38
Time 7 45 p. m.	2 3	2 46	·37	2·2	1·70	·20	2·20	1 90	·27	2·0	1·24	·27	3·75	3·0	·52
Time 8.3 p. m.	2·4	3·2	·30	3·3	1·67	·28	2·35	2·10	·44	2·14	1·50	·49	2·86	3·83	·62
Time 9.30 p. m	2·15	3 01	·43	1·9	2·01	·47	2·42	2·30	·58	2·14	1·33	·27	3·00	4·13	·53
Time 11 0 p. m. ..	2·30	..	..	..	..	..	2·50	..	..	..	..	..	5·00	..	..

TABLE VI

Model of River Indus and Kalabagh Headworks

RIVER DIVERSION

A comparative statement of the gauges obtained with different methods of river diversion

The alignment of diversion cuts was according to Mr. F. F. Haigh's diagram, dated 3rd April 1941

The diversion was carried out in a discharge of 31,000 cusecs.

Method No. 1—The right channel was closed first in a period equivalent to one month. After completely closing the right channel the central channel was closed in instalments which also took one month

Method No. 2—The central channel was closed first and the right channel later. The period required for closing the channel was the same as adopted in Method No. 1.

Method No. 3—The cuts were made of double the width than those used in Method No. 1 or 2. The central channel was closed first. The time of closing the channel used was same as before.

Gauge at Kalabagh Bridge			Nawab's bund			U/S of 1st diversion bund			D/S of 1st diversion bund			U/S of 2nd diversion bund			Intermediate No. 1		
1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	1	2	3
686·6	687·0	686·8	686·6	686·8	686·1	686·64	686·75	686·0	685·56	685·47	684·71	684·29	684·61	683·32	684·71	684·17	683·32
86·65	87·2	86·8	86·6	86·8	86·1	86·68	86·96	85·79	84·30	85·47	84·61	84·39	84·93	83·32	84·78	85·04	84·18
86·80	87·2	86·8	86·7	86·95	86·1	86·74	86·96	85·79	84·29	85·57	84·71	84·61	85·36	83·43	84·82	85·68	84·71
87·2	87·1	86·9	87·2	86·82	86·2	87·28	86·92	85·69	86·86	85·47	84·82	85·25	85·89	83·54	85·3	86·11	84·82
87·6	87·50	86·95	87·7	87·20	86·2	87·61	87·07	85·89	87·39	85·79	84·82	85·57	87·29	83·86	85·25	87·07	84·93
88·2	87·9	88·0	88·2	87·78	86·8	88·25	87·61	86·65	87·82	86·60	86·43	86·0	87·83	85·25	85·57	87·61	86·43
88·2	88·3	88·6	88·3	88·15	87·5	88·25	88·14	86·97	87·82	87·39	87·39	87·49	88·25	87·20	85·79	87·97	86·97
88·2	88·5	88·6	88·4	88·4	87·6	88·47	88·68	86·86	87·93	87·82	87·39	86·31	88·53	87·82	86·75	88·14	87·20
88·8	88·6	88·2	88·0	88·55	87·4	89·42	88·57	86·97	88·46	88·04	87·50	86·86	88·57	87·39	86·86	88·47	86·86
88·7	89·1	88·25	89·0	89·0	87·7	89·0	89·0	87·39	88·57	88·79	88·04	87·50	88·89	87·02	88·36	88·68	87·18
90·4	90·6	88·45	90·6	90·8	87·8	90·6	90·7	87·39	90·71	90·7	88·04	90·6	90·6	85·68	89·28	90·07	87·18
90·4	90·75	91·0	90·6	91·0	90·8	90·6	90·82	90·72	90·71	90·7	90·57	90·6	90·6	90·6	90·28	89·87	89·86

TABLE VI—concluded.

Intermediate No 2			D/S of 2nd diversion bund			U/S of 1st leading cut			U/S of 2nd leading cut			D/S of the leading cut			REMARKS
1	2	3	1	2	3	1	2	3	1	2	3	1	2	3	
682·37	682·37	683·80	681 93	683·80	Dry	684·71	683·75	682·14	681 07	681 70	Dry	675·2	675·6	675·4	1. Diversion done by closing the right channel first.
82·37	83·30	84 01	82 04	83·11	Dry	84 78	83·14	83·75	81 07	84·91	81·20	75·35	76·8	75·0	
82·37	83 79	84·71	82·04	83 47	Dry	84·92	83·47	84·39	81 07	84 82	81·39	75·4	76·0	75·5	2. Diversion done by closing the central channel first.
82 60	86·11	84 83	82 33	86 0	82 25	83 14	86 0	84 64	81 39	85·68	81·50	75·4	76·15	75·6	
82·80	87 01	84 93	82 66	86·96	82 36	85·14	86 86	84 86	81 25	86·79	81·61	75·5	76 2	76·0	3. Diversion done by closing the central channel first but the cuts made double width.
83 21	87 71	86 43	83 43	87 61	84 93	85 14	87·38	84 82	18 71	87·39	84·61	75 5	76·2	76·0	
85 99	88 03	86 97	83 79	87 93	85 89	85 14	87 71	85 65	85 35	87·89	85 46	76 4	76·4	76·4	
86 33	86 23	87 19	86 33	87 95	86 32	85 57	87 82	86·11	85 74	88 04	85·79	76·4	76 6	76·6	N. B.—Closing of each channel was done in six instalments. The bund was built every hour.
87 30	88 25	86 73	87 61	88 25	85·14	86·86	88 88	84·93	86·0	88·14	84·82	76·8	76·35	76·6	
88 13	88 47	87 18	87 30	88 14	85 35	82 07	85·36	84·93	87·37	88 47	85·14	76·2	77·2	76·8	
89 90	89 90	87 07	89 64	89 73	88 47	89 21	90·18	84 82	89·21	90·18	85·04	77·0	77·5	76·8	
89 98	89 64	89 33	89 00	89·64	87 18	89·21	89·64	86·73	89·21	89·64	86·32	77·0	77·5	77·2	

TABLE VII

River discharges above Suleimanke Headworks	Gauge at the nose of the earthe[n] bund
1	2
Cusecs	R. L.
1,000	571 ·5
2,000	572 ·1
3,000	572 ·6
5,000	573 ·3
7,000	573 ·7
8,500	574 ·1
10,000	574 ·3
3,000	572 ·6
5,000	573 ·3
8,500	574 ·1
10,000	574 ·3
12,500	575 ·6
15,500	575 ·8
8,500	574 ·2
20,000	575 ·5
30,000	577 ·3
50,000	578 ·5
60,000	578 ·8
75,000	579 ·0
100,000	579 ·5
75,000	579 ·1
100,000	579 ·6
125,000	580 ·3
150,000	580 ·8
200,000	581 ·0

TABLE VIII

River discharges above Suleimanke Headworks	Surface velocities
Cusecs	Feet
10,000	5 ·95
20,000	7 ·60
30,000	7 ·90
50,000	8 ·30
60,000	8 ·40
75,000	8 ·50
100,000	8 ·80
75,000	8 ·20
100,000	8 ·00
125,000	8 ·95
150,000	8 ·50
200,000	8 ·00

TABLE IX

River discharge above Suleimanke Headworks	Gauge at R. D. 39,000 — Right creek	Gauge at R. D. 39,000 — Central channel	Gauge at DD′ line — Right creek	Gauge at DD′ line — Central channel
1	2	3	4	5
Cusecs				
1,000	..	R. L. 569·9	..	..
2,000	..	R. L. 570·7	..	..
3,000	..	R. L. 570·9	..	..
5,000	..	R. L. 571·6	..	..
7,000	..	R. L. 571·9	..	..
8,500	..	R. L. 572·1	..	..
10,000	..	R. L. 572·2	..	..
3,000	..	R. L. 570·9	..	..
5,000	..	R. L. 571·6	..	..
8,500	..	R. L. 572·1	..	..
10,000	..	R. L. 572·2	..	..
12,500	..	R. L. 572·8	..	..
15,000	..	R. L. 572·5	..	..
8,500	..	R. L. 571·2	..	..
20,000	R. L. 571·2	R. L. 572·3	..	..
30,000	R. L. 571·0	R. L. 573·8	..	..
50,000	R. L. 573·2	R. L. 574·9	R. L. 571·7	R. L. 574·7
60,000	R. L. 573·8	R. L. 575·1	R. L. 572·3	R. L. 575·9
75,000	R. L. 575·2	R. L. 575·6	R. L. 574·8	R. L. 574·7
100,000	R. L. 576·1	R. L. 576·3	R. L. 575·2	R. L. 575·1
75,000	R. L. 575·9	R. L. 575·8	R. L. 575·5	R. L. 574·8
100,000	R. L. 576·4	R. L. 576·5	R. L. 575·8	R. L. 575·5
125,000	R. L. 576·7	R. L. 576·8	R. L. 576·2	R. L. 575·9
150,000	R. L. 577·1	R. L. 577·22	R. L. 576·5	R. L. 576·2
200,000	R. L. 578·0	R. L. 577·6	R. L. 576·9	R. L. 576·5

TABLE X

Model of the river Panjnad below Panjnad weir

HEAD ACROSS THE SPURS

Discharges	Spur at 3,500'								Spurs at 7,500'							
	Left spur		*Right spur*		*Difference*		*Difference of the level in the river*		*Left spur*		*Right spur*		*Difference*		*Difference of level in the river*	
	U/S of shank	D/S of shank	U/S of shank	D/S of shank	Left spur	Right spur.	Left spur	Right spur	U/S of shank	D/S of shank	U/S of shank	D/S of shank	Left spur	Right spur	Left spur	Right spur
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
Cusecs	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet	Feet
132,000	1·150	1 130	1 141	1·125	·02	·016	1·3	1 0	1·117	1 064	1·121	1·083	·053	·038	3·2	2·5
128,000	1·144	1·127	1·142	1·123	·017	019	1·0	1 1	1·120	1·064	1·115	1·080	·056	·025	3·3	1·4
77,000	1·121	1·096	1·107	1·087	·025	·020	1·4	1 3	1·078	1·076	1·076	1·034	·002	·042	·12	2·6
54,000	1·102	1·050	1·080	1·070	·022	·019	1·2	1·1	1·071	Dry	1·060	Dry	..	..	..	..

TABLE XI

Model of the river Panjnad below Panjnad Headworks

TWO SPURS AT 7,500′ BELOW WEIR

Discharge	Left			Right		
	U/S of shank	D/S of shank	Difference	U/S of shank	D/S of shank	Differen
Cusecs	Feet	Feet	Feet	Feet	Feet	Feet
56,739	1·099	1·027	·072	1·094	1·024	·07
			*4·1			*4·0
64,169	1·087	1·028	·059	1·084	1·043	·04
			*3·5			*2·5
80,356	1·097	1·037	·060	1·093	1·055	·03
			*3·5			*2·4
123,184	1·112	1·056	·056	1·112	1·076	·03
			*3·4			*2·3

*These give the head across the shank of the spur in the *actual*.

TABLE XII

Madhopur pocket model

SILT TEST WITH DIVIDE WALL AT PIER No. 4
PIER DISCHARGE =14,000 CUSECS, CANAL FULL SUPPLY.

Number of bays	Silt in Bays of Regulator	
	With divide wall	Divide wall removed
1	·0025	*Nil*
2	·17	·007
3	·20	·003
4	·25	·06
5	·29	·04
6	·25	·05
7	·42	·08
8	·54	·22
9	·12	·25
10	·34	·35
11	·30	·50
12	·34	·75
	Still pond in the area enclosed by divide wall.	

TABLE XIII

Experiments on A. P. M.

B = 0·8

SETTING AT BED LEVEL

Serial No.	Depth of water over the crest "H" (mean)	M. M. working head, hm	Coefficient C I	II	III	IV	V	VI	VII	Mean value of C
					$Y = 0·102$					
1	0·827	0·360	6·96	6·96	7·01	6·96	..	..	..	6·9
2	1·019	0·485	6·95	6·99	6·99	.	..	..	..	6·9
3	1·210	0·660	7·00	7·02	6·98	..	..	..	..	7·0
4	1·449	0·835	6·99	6·98	6·98	6·94	..	..	..	6·9
5	1·623	0·987	6·99	7·01	7·05	6·99	..	..	..	7·0
6	1·819	1·137	7·05	7·02	6·98	..	..	..	..	7·0
7	2·031	1·295	6·92	6·95	..	..	..	..	..	6·9
8	2·229	1·460	6·91	6·90	6·90	6·90	..	..	..	6·9
9	2·404	1·620	6·92	6·92	6·88	.	..	..	..	6·9
					$Y = 0·170$					
10	0·821	0·275	6·95	6·93	6·81	6·86	..	..	..	6·8
11	1·013	0·386	7·00	6·90	6·89	..	..	..	..	6·9
12	1·201	0·540	6·96	6·94	.	..	..	..	..	6·9
13	1·362	0·645	7·03	7·02	7·01	..	..	..	..	7·0
14	1·589	0·820	6·92	7·00	6·88	6·95	..	..	..	6·9
15	1·810	1·005	6·90	6·90	6·90	..	..	..	..	6·9
16	1·995	1·135	6·94	6·90	6·96	..	..	..	..	6·9
17	2·153	1·275	7·00	6·90	6·98	6·89	6·87	..	..	6·9
18	2·445	1·505	6·91	6·90	6·98	..	..	..	..	6·9
					$Y = 0·106$					
19	0·646	0·245	7·05	7·02	7·19	7·14	..	..	..	7·1
20	0·952	0·310	7·11	7·11	7·11	7·13	..	..	..	7·1
21	1·188	0·460	7·13	7·17	7·16	7·20	..	..	..	7·1
22	1·383	0·600	7·17	7·19	7·12	..	..	..	..	7·1
23	1·598	0·760	7·09	7·06	7·08	..	..	..	..	7·0
24	1·786	0·900	7·09	7·07	7·07	..	..	..	..	7·0
25	1·963	1·060	7·08	7·07	7·09	7·08	7·12	..	..	7·0
26	2·199	1·225	7·12	7·11	7·08	7·15	..	..	..	7·1

TABLE XIII—CONTINUED

Serial No	Depth of water over the crest "H" (mean)	M. M. working head, hm	Coefficient C I	II	III	IV	V	VI	VII	Mean value of C
			$Y = 0{\cdot}251$							
27	0·821	0·185	7·31	7·35	7·30	..	..	..	..	7·32
28	1·039	0·310	7·41	7·40	..	..	..	..	..	7·41
29	1·203	0·405	7·34	7·37	7·35	7·34	..	..	..	7·35
30	1·452	0·560	7·27	7·32	7·32	7·30	7·31	..	..	7·30
31	1·620	0·670	7 31	7·36	7·31	7·39	..	..	..	7 34
32	1·817	0·813	7 30	7·26	7·29	..	..	..	..	7·28
33	2·020	0·955	7·26	7·26	7·26	7·24	..	..	..	7·26
34	2 233	1·130	7·30	7·29	7·31	7·36	..	..	—	7·32
35	2·396	1·270	7·28	7·25	7·24	..	..	..	..	7·26
			$Y = 0{\cdot}300$							
36	0·781	0·150	7·24	7·26	7·24	7·23	..	..	..	7·24
37	0 997	0·230	7·27	7·30	7·29	7·35	..	..	..	7·24
38	1·191	0·350	7·36	7·40	7·37	..	..	..	..	7·38
39	1·392	0·480	7·34	7·37	7·37	..	..	..	..	7·36
40	1·575	0·575	7·28	7·27	7·27	7·26	..	..	..	7·27
41	1·781	0·747	7·35	7·38	7·32	7·33	..	..	..	7·35
42	1 951	0·840	7·33	7·33	7·31	7·30	..	..	..	7·32
43	2 181	1 040	7·33	7·37	7 33	7·36	..	..	..	7·35
44	2·376	1·190	7·34	7·34	7·35	..	..	.	..	7·34
			$Y = 0{\cdot}351$							
45	1 022	·215	7 31	7·32	7 29	..	.	..	..	7·31
46	1·173	0·300	7·34	7·37	7·35	7·37	..	..	..	7 36
47	1·375	·405	7·40	7·36	7·37	7 37	..	..	..	7·38
48	1·615	0·555	7·27	7·26	7·29	..	..	..	.	7·27
49	1·837	0·685	7·25	7 26	7·23	7·28	..	..	..	7·26
50	1 936	0·755	7·26	7·25	7·24	7·24	7·25	..	.	7·25
51	2·221	0·980	7·30	7·30	7·30	7·29	..	.	..	7 30
52	2·427	1·145	7·31	7 32	7·31	7 30	.	.	.	7 31

TABLE XIII—concld.

Serial No.	Depth of water over the crest "H"(mean)	M.M. working head, hm	Coefficient C I	II	III	IV	V	VI	VII	Mean value of C
			Y = 0·401							
53	·991	·185	7·30	7·35	7·29	..	..	..	..	7·31
54	1·175	·263	7·46	7·46	7·50	..	..	..	..	7·47
55	1·365	·390	7·46	7·47	7·46	..	..	..	..	7·46
56	1·591	0·490	7·35	7·35	7·40	7·40	..	..	..	7·38
57	1·813	0·630	7·40	7·42	7·40	7·38	7·40	..	..	7·40
			Y = 0·467							
58	·751	0·085	7·41	7·45	7·49	..	..	..	..	7·45
59	1·010	0·190	7·14	7·13	7·14	..	..	..	..	7·14
60	1·215	0·250	7·36	7·35	7·36	..	..	..	..	7·36
61	1·413	0·350	7·44	7·44	7·42	..	..	..	..	7·43
62	1·603	0·450	7·24	7·24	7·24	7·22	..	..	..	7·24
			Y = 0·499							
63	1·208	0·240	7·22	7·26	7·25	7·26	7·26	7·22	..	7·25
64	1·407	0·340	7·37	7·35	7·35	..	..	..	..	7·36
			Y = 0·535							
65	1·000	0·106	7·05	7·05	7·05	7·04	..	..	..	7·05
			Y = 0·557							
66	1·050	0·185	6·9	6·93	6·93	6·92	6·92	..	..	6·93
67	1·202	0·225	..	7·20	7·20	7·20	7·23	7·21	..	7·21
68	1·422	0·255	..	7·39	7·43	7·43	7·40	..	..	7·41

TABLE XIV

Experiments on A. P. M.

B = 1·00', SETTING AT BED LEVEL

Serial No.	Depth of water over the crest "H"	M. M. working head, hm	Coefficient I	II	III	IV	V	VI	Mean value of C_2
				$Y = 0·104$					
1	0·802	0·35	7·11	7·15	7·13	..	–	–	7·13
2	0·994	0·495	7·21	7·18	7·21	..	..	–	7·20
3	1·184	0·625	7·18	7 19	7·16	..	..	–	7·18
4	1·424	0·815	7·21	7·22	7·21	..	..	–	7·21
5	1·594	0·925	7·16	7·16	7·15	7·16	..	–	7·16
6	1·800	1·110	7·16	7·20	7·15	7·14	..	–	7·17
7	2·003	1·275	7·17	7·16	..	7·19	..	..	7·17
8	2·202	1·445	7·17	7·17	7·17	7·19	..	–	7 18
9	2·379	1·610	7·12	7·12	7·10	7·12	..	–	7·12
				$Y = 0·132$					
10	0·771	0·263	7·19	7·29	7·25	..	–	–	7·24
11	0·998	0·436	7·30	7·25	7·28	7·28	–	–	7·28
12	1·171	0·555	7·33	7·30	7·32	..	–	–	7·32
13	1·316	0·650	7·33	7·31	7·31	..	..	–	7·32
14	1·565	0·845	7·20	7·22	..	..	–	–	7·21
15	1·817	1·045	7·16	7·16	7·16	7·18	..	..	7·17
16	1·975	1·165	7·16	7 16	7·16	7·16	..	..	7·16
17	2·131	1·285	7·24	7 18	7·19	7 19	..	..	7·20
18	2 409	1 535	7·16	7 16	..	..	..	..	7·16
				$Y = 0 135$					
19	·842	·340	7·36	7 31	7·38	..	..	..	7·35
20	·981	·407	7·41	7·33	7·28	..	..	..	7·34
21	1·226	·570	7·36	7 35	7·34	7 37	..	..	7·35
22	1·401	·705	7·35	7 31	7·31	7 31	..	..	7·32
23	1·581	·842	7·25	7 25	7·35	..	..	..	7·34
24	1 782	1·025	7·31	7·32	7 38	.	.	..	7 34
25	1 957	1·176	7 29	7 38	7 37	7·33	..	..	7·34
26	2·200	1·339	7·26	7·25	7·22	7·31	..	..	7·2
27	2·427	1·550	7·30	7·28	7·32	..	..	..	

TABLE XIV—continued

Serial No.	Depth of water over the crest "H"	M. M. working head, hm	Coefficient I	II	III	IV	V	VI	Mean value of C
			$Y = 0·199$						
28	0·924	0·300	7·12	7·11	7·16	7·11	..	..	7·13
29	1·153	0·430	7·05	7·04	7·08	7·10	7·13	..	7·08
30	1·356	0·570	7·08	7·10	7·10	7·06	7·06	..	7·08
31	1·564	0·720	7·10	7·10	7·09	7·05	7·05	..	7·08
32	1·751	0·875	7·13	7·11	7·15	..	..	..	7·13
33	1·931	0·900	7·11	7 08	7·13	7·11	..	..	7·11
34	2·171	1·185	7·08	7 07	7·12	7·13	7·10	..	7·10
35	2·309	1·295	7·07	7·09	7·14	..	--	..	7·10
			$Y = 0·251$						
36	1·009	0·270	7·44	7·44	7·43	7·46	--	--	7·44
37	1·175	0·375	7·45	7·45	7·45	7·46	--	..	7·45
38	1·420	0·525	7·43	7·40	7·42	--	..	--	7·42
39	1·585	0·650	7·51	7·49	7·47	--	..	..	7·49
40	1·772	0·770	7·51	7·48	7·48	7·48	--	--	7·49
41	1·988	0·920	7·44	7·44	7·44	7·41	--	--	7·43
42	2·204	1·105	7·36	7·41	7·39	7·42	..	--	7·40
43	2·361	1·230	7·44	7·44	7·45	7·45	..	..	7·45
			$Y = 0·307$						
44	1·166	0 325	7 35	7·32	7·29	7·35	..	..	7·33
45	1·361	0 430	7·39	7·35	7·37	7·37	7·35	..	7·37
46	1·542	0·545	7·30	7·37	7·31	7·31	7·44	..	7·36
47	1·751	0 670	7·45	7·44	7·45	7·44	..	..	7·45
			$Y = 0·355$						
48	0·993	0·190	7·30	7·39	7·34	7·36	..	..	7·35
49	1·149	0·272	7·36	7·35	7·30	7·30	--	--	7·33
50	1·338	0·350	7·41	7·42	7·41	7·40	..	..	7·41
51	1·582	0·525	7·39	7·39	7·41	7·41	..	..	7·4

TABLE XIV—CONCLUDED

Serial No.	Depth of water over the crest "H"	M. M. working head, hm	Coefficient I	II	III	IV	V	VI	Mean value of C
				Y = 0·357					
52	·819	·145	7·29	7·26	..	..	..	..	7·28
53	·983	·200	7·33	7·39	7·31	..	..	..	7·34
54	1·203	·325	7·39	7·39	7·41	..	..	..	7·40
55	1·406	·410	7·35	7·36	7·42	..	..	..	7·38
56	1·590	·515	7·33	7·33	7·31	..	..	..	7·33
				Y = 0·405					
57	1·121	·225	7·46	7·50	7·43	7·54	..	..	7·48
58	1·338	·355	7·46	7·46	7·41	..	..	..	7·44
				Y = 0·455					
59	0·788	0·170	7·25	7·35	..	..	..	..	7·30
60	0·970	0·150	7·21	7·22	7·23	7·20	..	..	7·22
61	1·167	0·215	7·32	7·32	7·32	7·32	7·32	..	7·32
62	1·412	0·360	7·30	7·34	7·36	..	..	..	7·33
				Y = 0·506					
63	1·020	·175	7·15	7·15	7·15	..	..	..	7·15
64	1·170	·190	7·31	7·30	7·27	..	..	..	7·29

FIG. 180

APPARATUS FOR NEGATIVE PRESSURE

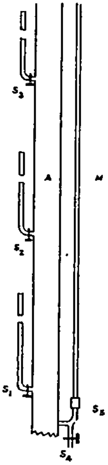
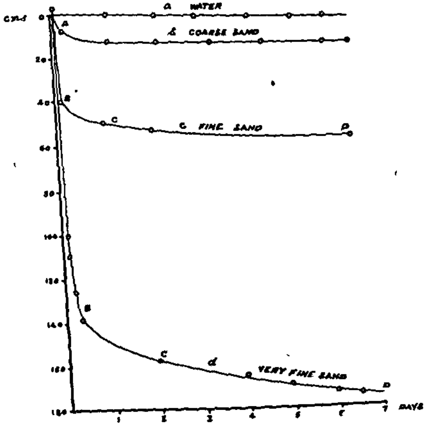


FIG. 181

DEPRESSION OF WATER-TABLE IN SAND



Physics Section.

Movement of Water-level in Pipes and its Relation to Water-table—It was reported last year that experiments were in progress to study the effect of evaporation and drainage on subsoil water-level. These experiments were further developed in the laboratory.

An experimental tube A, closed at the bottom, 200 cm long and 8 cm in diameter, was fitted with a number of side tubes S_1, S_2, S_3, S_4 and S_5. These are shown in Fig. 180. The tube is packed with the sand to be investigated under water. Great care in packing is essential to obtain consistent results. Water free from air bubbles and from suspended matter, is allowed to flow down the sand column continuously for about three weeks to complete the settling. The pressure gradient as given by the pressure pipes S_1, S_2 and S_3 indicates the uniformity of packing along the length of the tube. The value of the transmission constant is determined at intervals and when it becomes constant shows that the sand column has been stabilised. The inflow of water is then stopped. S_1, S_2, S_3 and S_4 are closed while S_5, which connects the main tube with the glass tube M, is opened. The level of the water in M now rises to the level of the free water in A. Evaporation is allowed to take place from the sand surface under atmospheric conditions. So long as there is any free water present above the sand surface, the level of water in M falls very slowly remaining the same as that in A, but as soon as the water level in the pipe touches the sand surface the level in M begins to fall very rapidly. The level in M corresponding to the commencement of the rapid rate of fall is taken as the zero datum level.

In the first set of experiments three specimens: coarse, fine and very fine, of sand were used. The experimental cylinder was packed with each specimen in turn and the readings of the side tubes were observed, when evaporation was taking place from the surface of the sand, the sand being initially saturated.

The results are shown in Fig. 181. Referring to Fig. 181, it will be seen that there are four curves in it, viz., *a*, *b*, *c* and *d*. The curve *a* represents the rate of evaporation from the surface of water. Curve *b* shows the rate of depression of water-level in the pipe with respect to time, for very coarse sand. There is an initial rapid fall of about 5 cm and subsequently the rate of fall becomes very slow. The sand particles in this experiment were between 2·5 mm and 3 mm.

Curve *c* shows the rate of depression of water-level in the pipe for fine sand as indicated by the level in M. In this case the rapid initial fall continues up to about 10 cm and subsequently the rate becomes very slow. The curve is composed of three distinct parts, AB, BC and CD. The fall along AB is very steep but the rate of fall decreases in the region BC. From C to D the curve becomes almost asymptotic to the time axis showing that the rate of depression is now

becoming very small. A mechanical analysis of this sand was made and the size distribution curve is shown in Fig. 182.

Curve *d* in Fig. 181 shows the rate of depression of water-level in the pipe for very fine sand. The nature of the curve resembles that for fine sand, though the numerical values are of course different. In this case, the initial steep fall continues up to 150 cm and then the rate decreases. The mechanical analysis of this specimen of sand is given in Fig. 183.

These results are of practical importance. They show that in the initial stages, a certain amount of water, say Q, lost by evaporation produces a depression in the water-level in the pipe much more than at later stages. In the usual method of calculation, if A is the effective area of cross section of the cylinder, the depression in water-level in the pipe will be equal to Q/A, for a loss of quantity Q of water. If the pore space is about 40 per cent the effective area A is equal to 40 per cent of the area of cross section of the cylinder. For very coarse sand, the initial fall is about 10 times that calculated from Q/A, while for fine and very fine sand, these are about 80 and 250 times respectively. The fall of water-level in the pipe is thus dependent upon particle size, under similar conditions of evaporation.

The water-level in a pipe attached to a sand-water column is thus controlled by the particle size. If the same amount of evaporation can cause different changes in the level of water in the pipes, then it is clear that a water level in a pipe is not directly related to the quantity of water lost by evaporation. Past attempts, therefore, to connect the fall of water-level with evaporation were incorrect. The pipe under such conditions measures only the pressure deficiency at the air-water-soil interface near the natural surface of the soil. This deficiency depends upon the curvature of the water menisci at the top of the capillary column and, hence, on particle size. This is the reason why no direct relationship exists between the fall of water-level in the pipes and the amount of evaporation.

Keen obtained a steep fall of water-level in pipes in which he had carried out experiments lasting for six years. Keen gave an explanation of the steep initial fall in free water-level in those pipes, based on the experiments of Haines. This explanation and the experiments on which it was based were not in accordance with our laboratory experiments. Experiments were therefore started in order to re-investigate Keen's explanation and Haines' experiments.

Experiments on Pressure Deficiency with Porous Pots—It was reported last year that experiments were being carried out on pressure deficiency and moisture content on various grades of soil. The arrangement used in the experiments under report is essentially similar, but refinements were introduced in the apparatus to enable a series of readings to be taken on pressure deficiency, very near saturation.

FIG. 182

FINE SAND

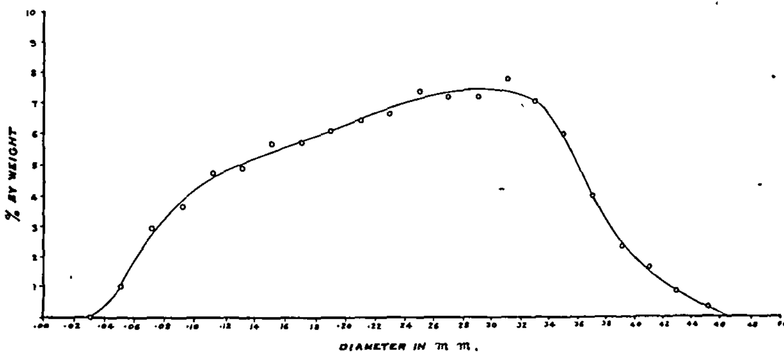


FIG.183

VERY FINE SAND

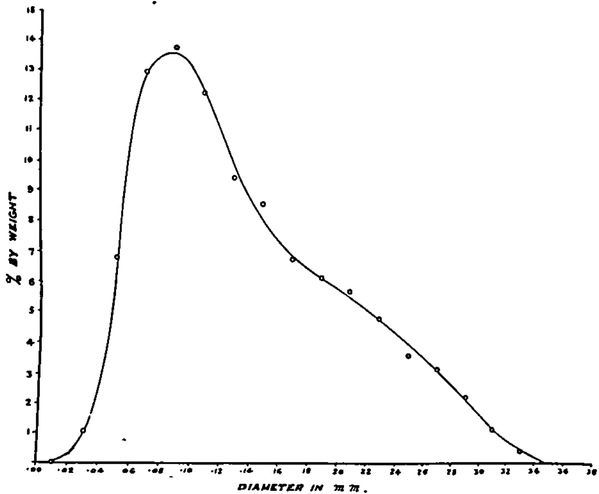


FIG. 184

ARRANGEMENT FOR PRESSURE DEFICIENCY

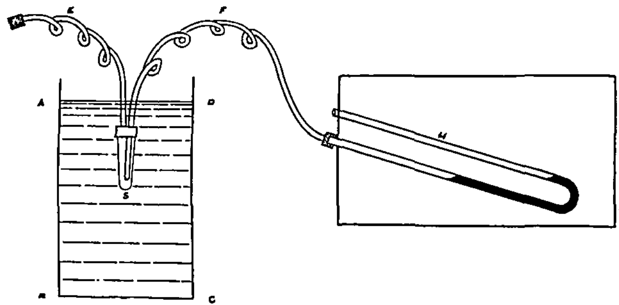
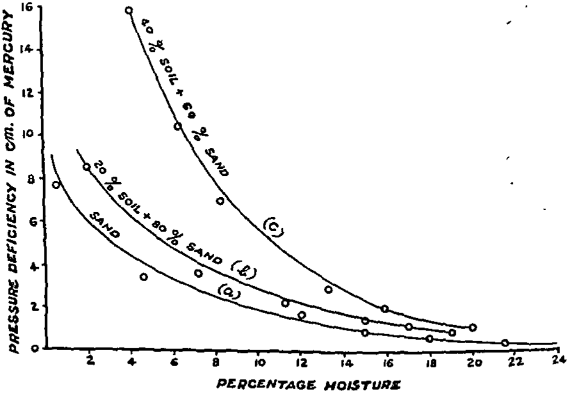


FIG. 185

PRESSURE DEFICIENCY NEAR SATURATION



The apparatus to test this is shown in Fig. 184. ABCD is a glass vessel containing the sand to be investigated. S is a porous pot about 2 inches in length and 1 inch in diameter. E and F are copper pipes $\frac{1}{4}$ inch in diameter and soldered to a brass cap. The brass cap fits the mouth of the porous pot. The copper pipe, E, serves to fill the porous pot with water and the other pipe,-F, is connected to an inclined mercury manometer M as shown in the figure. The manometer can be given any angle so that the desired sensitivity may be obtained. A travelling microscope was focussed on to the mercury meniscus to enable the readings to be taken accurately.

The readings obtained with saturation and pressure deficiency for three specimens of sand are plotted in Fig. 185. It will be seen from *the figure that there is no abrupt change of pressure deficiency,* from saturation down to 2 per cent moisture. A number of observations were taken very near saturation but in none of them was a rapid initial fall obtained. Keen's explanation was, to quote his own words, "that from a consideration of the suction pressure as developed by Haines the rapid initial fall is evident. Haines' curve for the relation between pressure deficiency and moisture content of the soil shows that just below saturation the moisture content changes very slightly for an appreciable increase in pressure deficiency above its initial value at saturation." Haines experimented on glistening dew. The results obtained with glistening dew do not apply to sands.

The correct explanation for any such initial rapid fall is to be found not in unsaturation but in *the development of the negative* pressure due to the curved micro-menisci at the top of the capillary column. We use the term negative pressure here for a pressure less than that at the phreatic surface. This is to avoid confusion with the term pressure deficiency which may be associated with partial saturation. A test of the meniscus theory was applied in a series of experiments which are described subsequently.

If the water-level in a pipe embedded in a saturated sand column is controlled by capillary forces at the soil-air-water interfaces, then what would be *the effect if evaporation takes place in a stratified* soil? If, for instance, a coarse sand with a layer of finer sand at the top is placed in the cylinder and initially saturated, a large negative pressure will be developed as evaporation begins, because the finer sand exerts a larger negative pressure at the soil-air-water interfaces than a coarse sand. A pipe connected to the cylinder of sand will show a rapid fall. As evaporation proceeds and the air-water interfaces are brought into the coarser layer, the negative pressure must decrease, because, as stated, a coarse sand exerts less negative pressure at the micro-interfaces than fine sand. If the water-level in a pipe is only a record of this pressure, as has been found, then the water in the pipe will rise though water is still being lost by evaporation. This may sound anomalous but was confirmed in the following experiments.

Experiments on Negative Pressure and Stratification—The experimental arrangement and the method of conducting the investigation have already been described. The difference in this case is that the homogeneous column of sand is replaced by two layers, the top one being composed of fine sand and the bottom one of coarse sand. Experiments were carried out with different thicknesses of the top layer.

Fig. 186 shows the effect of evaporation on the movement of water-level under the conditions mentioned in the above paragraph. The four curves in the figure are for different thicknesses of the top layers. The thicknesses of the layers varied from ·5 to 10·5 cm.

In all the four cases investigated there is an initial steep fall of level. This fall then ceases and a rise commences. Finally after a series of reversals the water-level in the pipe attains an almost steady state. In the case of the thinnest layer, this state is attained in about three hours, whereas with the thickest layer to acquire a steady level takes about three days. The final steady level is attained when the level, after the first fall, has risen to about ten centimeters below the surface of the coarse sand. This latter level is characteristic of the lower coarse stratum only. The composition of the top layer does not seem to affect this final steady level, though the initial fall is controlled by the top layer. It will also be noticed that the rise of water-level is not smooth, but is attained by a series of quantum-like movements.

Fig. 187 shows the results of experiments similar to those referred to in Fig. 186, but here the top layers are composed of finer sand than used in the experiments referred to in the previous paragraphs. The initial fall of level is greater than that referred to before, but in other respects the curves are interpreted similarly.

These experiments conclusively show that it is the negative pressure caused at the top of the capillary column that is responsible for the maintenance of the level of water in the pipe. The theory that it is the negative pressure of the meniscus at the top of the capillary column which controls the level in the pipe is completely verified by the reversal of the level in these experiments. It is wrong to take this level as water-table; or if we take this level as water-table, then this water-table will bear no relation to the amount of water lost from the water-table by evaporation.

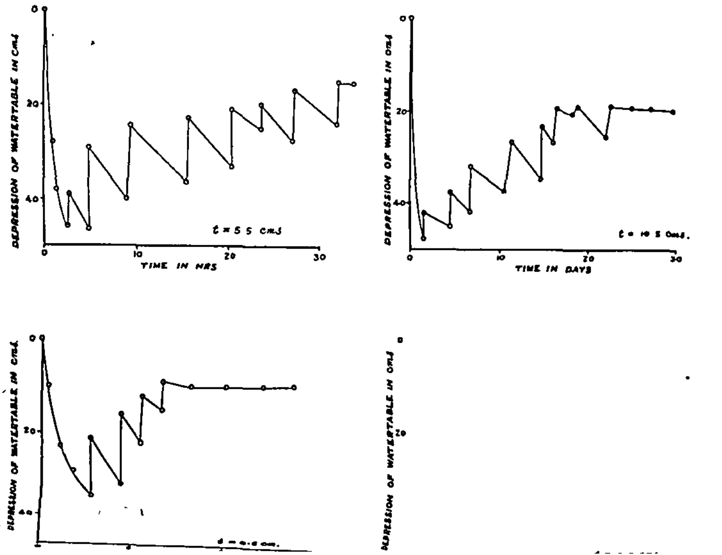
The bearing of these experiments on the measurements of water-table will be discussed subsequently.

The next series of investigations carried out in the laboratory were pertaining to the effect of this new phenomenon on the drainage of soils.

Effect of Capillarity on Drainage in Waterlogged areas—The fact that a capillary negative pressure of considerable magnitude is caused by the menisci at the soil-air-water interfaces on the top of the

FIG.

VARIATION OF NEGATIVE PRESSURE WITH FINE SAND AT TOP



VARIATION OF NEGATIVE PRESSURE WITH VERY FINE SAND AT TOP

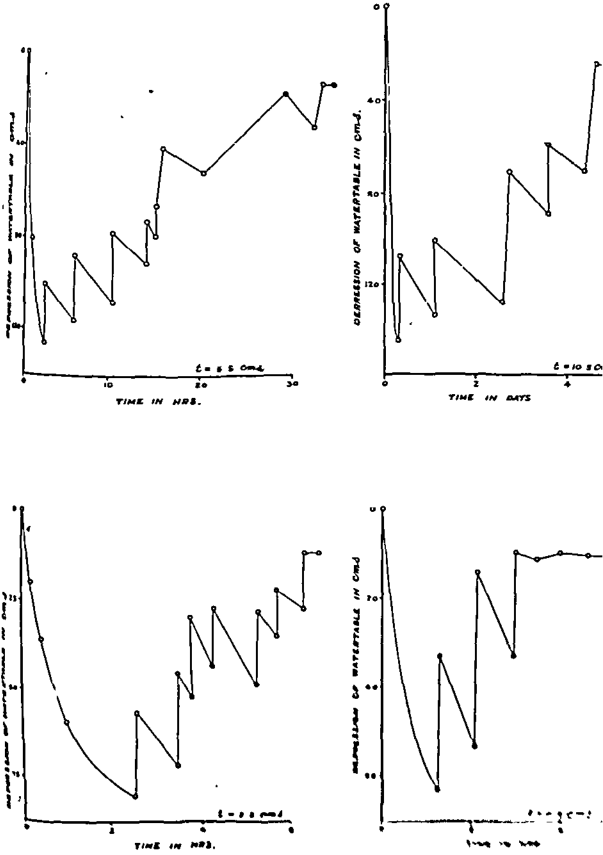


FIG. 188

VERY COARSE SAND

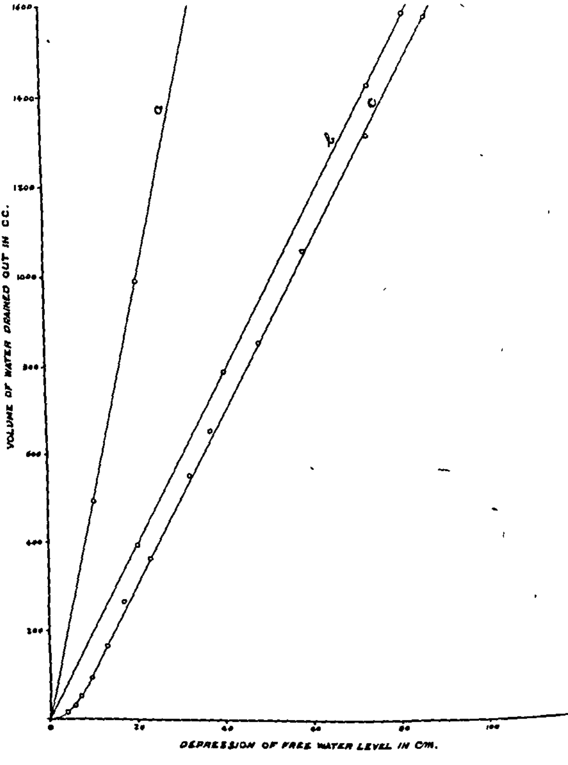


FIG.189

FINE SAND

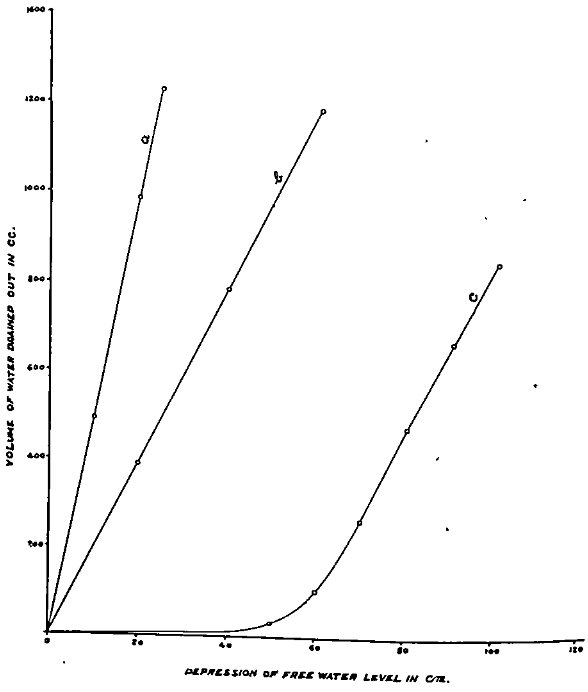
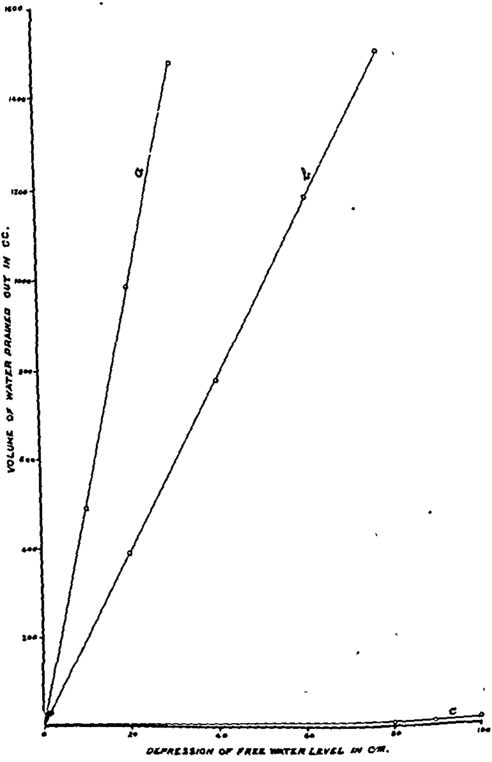


FIG. 190

VERY FINE SAND



capillary column has an effect on the drainage of soils in waterlogged areas.

A cylinder similar to that shown in Fig. 180 was fitted with four side tubes. The lowest side tube has a long tube of glass of small bore attached to it. This tube is as high as the cylinder and the fluctuations of water-level in the cylinder are observed in this glass tube.

Before beginning an experiment, water is allowed to pass down the cylinder for two weeks in order to stabilise the conditions. Free water is then allowed to stand on the sand and the level of water in the side tubes is noted. One of the side tubes, say the second from the bottom, is now opened so that water begins to drain. As water is being drained the level of water in the cylinder and in the side tube remains the same as in a U tube so long as there is free water above the sand. When the drainage has reached such a stage that this free water surface touches the sand surface, the level of water in the side tube sinks very rapidly. The subsequent depression bears no direct relation to the amount of water drained, but it depends on the particle size of the sand in the cylinder. Experiments were carried out with three different grades of sands, and the results are shown in Figs. 188, 189 and 190.

Fig. 188 is for very coarse sand. Curve *a* in Fig. 188 shows what the depression should be if there was no sand in the cylinder and if both the cylinder and the side tube were only filled with water. Curve *b* shows the theoretical depression if the cylinder is packed with sand under water and drained. Here it is assumed that the sand has 40 per cent pore space and if the water is completely drained, the water level in the cylinder and in the pipe should fall along curve *b*. In the actual experiment, the curve traced was *c*.

After an initial stage the curves *b* and *c* run parallel. The horizontal separation between *b* and *c* shows that the level of water in the pipe is lower than that in the cylinder. In the stage corresponding to the parallel portions of the curves, the fall of water-level in the pipe reflects correctly the fall in the water-level in the sand column. There can, therefore, be no serious error if the amount of drainage is correlated in this case with the fall of water-level in pipes.

The case of a fine sand will now be examined. The results are shown in Fig. 189. Curves *a*, *b* and *c* have here the same significance as in Fig. 188. The result of this finer grade sand is that curves *b* and *c* are separated very much more than in the previous case.

For the sake of convenience in discussion this curve *c* may be split up into three parts MN, NP and PQ. In the part MN, the curve is almost parallel to the abscissa ; in the part NP the slope increases rapidly and after P the curve runs parallel to *b*.

It will be seen from a comparison of Figs. 188 and 189, that the part corresponding to MN was very small in the former case whereas it is very large in the latter case. This shows that in the initial stages

of draining of a soil, a very large depression is caused for a small amount of water drained and that this difference becomes greater as the soil becomes finer. Thus in Fig. 189*c* when 50cc of water are drained, a depression of 20cm is observed in the pipe, whereas if the same amount of depression is to be produced in the water surface in the cylinder 400cc of water should be drained out. This looks anomalous but is an experimental fact. In the region corresponding to NP smaller quantities of water are required to be drained to produce the same depression as in MN. After the stage corresponding to P is attained the curve runs parallel to *b*. Here the amount of water drained corresponds to the fall in level of water in the pipe just as one might expect, i.e., if Q is the quantity of water [illegible] area of cross section, then the fall of wa[illegible] correspond to that in the pipe. Here the [illegible] is 40 per cent of that of the cylinder, this being approximately the void space in sand. The divergence between *b* and *c* is very much greater than in the corresponding case of Fig. 188.

In Fig. 190 the stage corresponding to N could not be obtained with a height of 150 cm and this was the maximum range of the experimental tube. The stage corresponding to PQ will be attained after 7 feet. In this experiment, the sand used was very fine.

These results have an important bearing on drainage and the measurement of water-table.

Seepage drains in the Punjab vary in depth from three to ten feet and are dug in high water-table areas. They are meant to deal with storm and subsoil water.

The new phenomenon observed in the laboratory has no bearing on the removal of storm water. When the storm water has been drained and the surface of the soil is exposed to air, the soil-air-water interface develops a negative pressure depending on the grain size of the soil. In the case of very fine sand the negative pressure which this can develop is seen to be seven feet. Unless the depth of the drain exceeds seven feet, the pores in the soil must remain full of water, though a pipe embedded in the soil may record a water depth of seven feet. A very insignificant quantity of water removed by the drain is enough to bring into operation the negative pressure of the micro-menisci at the top of the capillary column, and thus show an *apparent* decrease in the water-level in the pipe. In the case of very fine sand referred to in Fig. 190, a drain, for example, say, three feet in depth need only remove less than 1/180th of the water occupying the pore space to show a depression of three feet in a pipe embedded in the sand. More water cannot be removed by this depth of drain, because a negative pressure equal to three feet has already been developed at the micro-interfaces of the soil-air-water on the top of the capillary column. Since this negative pressure is about seven feet for this sand, conditions of saturation will remain the same until this critical depth of the drain is passed.

What is probably taking place in a field is that when the surface water is removed, evaporation takes place from the surface of the soil and any subsequent reduction in moisture content of the soil above the water-level in the drain will be brought about by evaporation only. It is not possible in a report to discuss in detail the conclusions and their bearings on problems in irrigation. For these, reference may be made to the paper published on the subject. The main conclusions from these experiments on evaporation and drainage carried out in the laboratory may however be summarised. It may be pointed out here that these conclusions apply to high water-table areas. It was mentioned earlier that the conclusions would be summarised after describing the drainage experiments. This was convenient because the effect of evaporation or drainage of the soil had many points in common and the conclusions therefore apply to both.

(1) The first effect of evaporation or drainage of a soil is to remove the free water standing on the soil. The capillary forces of the soil have no effect on this stage.

(2) When the free water on the surface has been evaporated or drained and the soil is exposed, the first effect is to cause the development of the micro-menisci at the soil-air-water interface.

(3) As a result of these micro-menisci, a negative pressure is developed and a pipe embedded in a soil begins to record this negative pressure in the manner of a manometer.

(4) Since this negative pressure is characteristic of the particle size of the soil, the pipe record shows a much larger depression in finer soils than in coarser ones and this record therefore has no relation to the quantity of water lost either by evaporation or by drainage.

(5) If water is added to the soil surface at this stage, a small quantity will produce an abnormal rise because the effect of this addition is to destroy the negative pressure. The rise of water in a pipe will be out of proportion to the quantity of water added.

(6) Attempts which have been made in the past to correlate the rise of water-level in pipes with the amount of irrigation and seepage were therefore incorrectly conceived. No such attempt can be successful unless the forces caused at the micro-menisci on the top of the capillary column are taken into account. This explains the abnormalities of well records, which have very often been attributed to careless readings.

(7) Seepage drains cannot reduce the moisture content of a soil much below its saturation, unless the depth of the drain is beyond a critical depth which can be determined by experiment.

(8) It is possible that when the surface water is removed by shallow drains, evaporation is the only factor controlling the moisture content of the surface soil layer.

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(7) Seepage drains cannot reduce the moisture content of a soil much below its saturation, unless the depth of the drain is beyond a critical depth which can be determined by experiment.

(8) It is possible that when the surface water is removed by shallow drains, evaporation is the only factor controlling the moisture content of the surface soil layer.

(9) The conception of a water-table as that corresponding to the phreatic surface recorded by a pipe is of no value, because the phreatic surface moves up or down according to changes of pressure at the top of the capillary column and gives no idea of the amount of water added to or removed from the subsoil. The phreatic surface is not an air-water surface within the subsoil.

(10) The air-water-surface in the subsoil is in a plane higher than the phreatic surface and it is the former that responds quantitatively to the amount of water added by seepage or removed by evaporation. If a relation between the amount of water added or lost from the soil and the corresponding movement of the water surface is to be sought, then it is the latter surface and not the phreatic surface in a pipe that should be taken as the water-table.

(11) If the strata are not uniform, then the forces at the interfaces of the strata are different from those in any one of them and when the air-water-surface passes from one stratum to another, say from finer to coarser, a falling phreatic surface will begin to rise though water is still being drained from the subsoil. This makes it clear that it is the force of capillarity that controls the water-level in a pipe.

(12) Since the grain sizes of the soil in any area are not uniform, the water-table, i.e., the surface below which the pores are full, will also not be in a horizontal plane. Though there are difficulties in finding out the position of this surface, this is not a reason for continuing to regard the phreatic surface as the water-table.

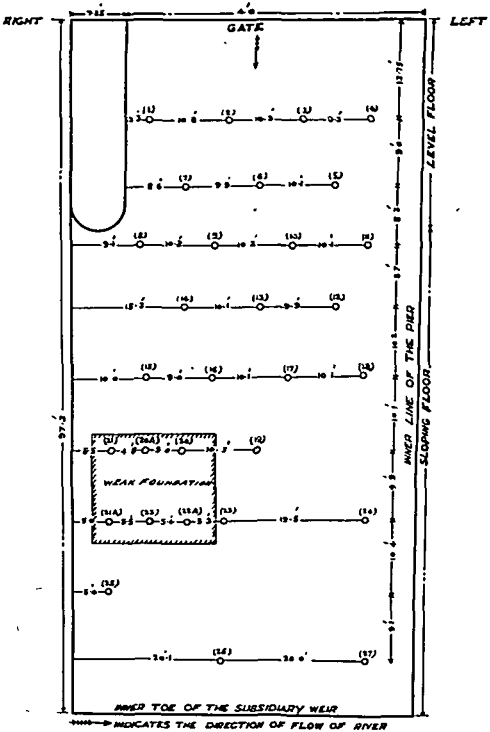
(13) When the water-table is very deep, these considerations are not of any great significance.

Experiments on Cavities in Balloki Weir Bay No. 1—It was reported last year that experiments were in progress to locate cavities under the Balloki Weir. The method adopted was to use the grouting pipes as electrodes and map out the resistances between the pipes, by employing an alternating current. The apparatus used consists of an A. C. source and a resistance measuring bridge equipment. The arrangement of the apparatus and the method of carrying out the experiments have already been described in that report, and reference may be made to this for details. Results for Bay No. 2 were reported last year. Experiments were carried out in Bay Nos. 1, 3, 16, 17, 18, 30 and 31 during the year under report. Except in Bay No. 1 the number of grouting pipes was small and so the results were insufficient to come to any definite conclusion. The results reported now are for Bay No. 1. The plan of the bay is shown in Fig. 191. This bay has about thirty pipes and six series of measurements were taken. Three series of observations were taken by connecting the pipes across the weir and another three series were taken by connecting the pipes along the weir. These wer[illegible]
The resistances between a[illegible]
may be due to the fact that [illegible]

FIG. 191

BALLOKI WEIR

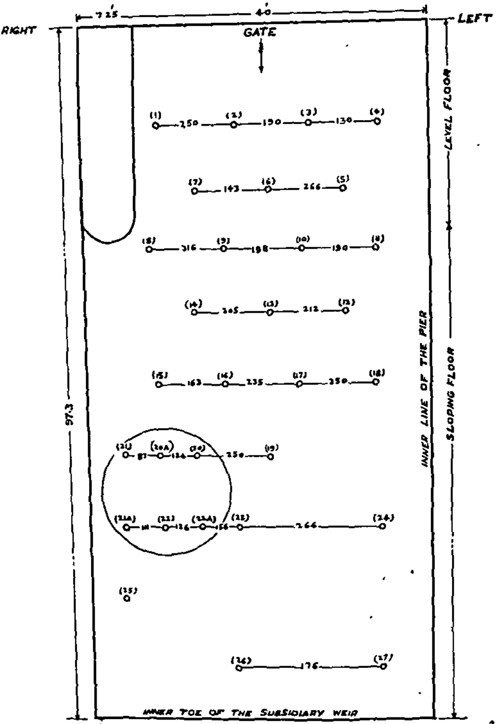
PLAN OF BAY No. 1 SHOWING THE POSITION OF PIPES



→ INDICATES THE DIRECTION OF FLOW OF RIVER

○ INDICATES GROUTING PIPES

FIG. 192
BALLOKI WEIR BAY No. 1

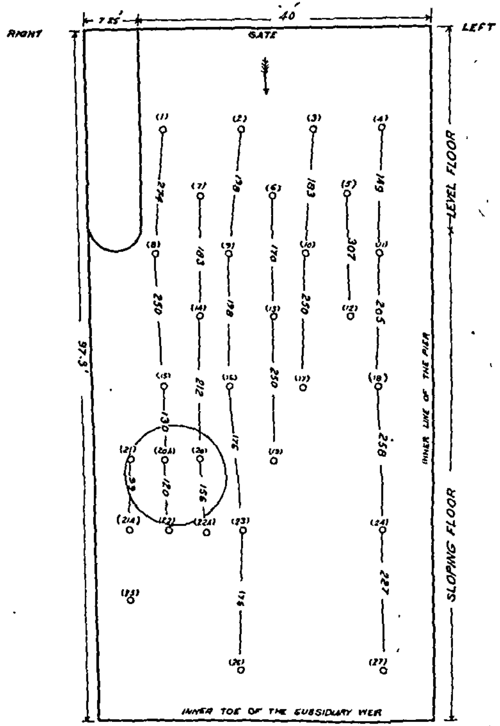


→ INDICATES THE DIRECTION OF FLOW OF RIVER
O INDICATES GROUTING PIPES

25.3.41

FIG 193

BALLOKI WEIR BAY No. 1



→ INDICATES THE DIRECTION OF FLOW OF RIVER

O INDICATES GROUTING PIPES

25 3 41

a weir or it may be due to some other cause at present not understood. Each of these observations were examined and a comparatively small resistance was obtained in the region of pipes 21 to 23 as marked in Figs. 192 and 193. Thus, referring to Fig. 192 while the maximum resistance obtained is about 250 units, the pipes in the region marked in the circle give only a range between 87 and 156 units. Referring to Fig. 193 where the measurements were taken by connecting pipes along the weir, it will be seen again that while a maximum resistance of about 300 units is obtained, in the region of pipes 21 to 23 marked in the figure the resistance varies from 99 to 150 units only. This again happens to correspond to the region referred to in Fig. 192. The other four sets of readings are not shown in the figures, but in all of them this particular region showed a low resistance. The cumulative evidence of all the sets of measurements points to the conclusion that this part of the weir has a weak foundation. It is purely a matter of technical terminology whether we call it a cavity, weak foundation or loose packing of the sand under the floor. That there is something [illegible] o be clear from the experiments. [illegible] ing Engineer, Lower Bari Doab Canal in his report on grouting. He stated that "on comparing the experimental results with his plan of those obtained by grouting, it was evident that the experiment had located an area, where there was a considerable concentration of channels."

Kalabagh Weir. Electrical Analogy Method for Detection of Cavities—A problem arose in connection with Kalabagh Weir, now under construction. Due to pumping the pressure pipes in the weir showed readings very different from those based on the design. A model of the weir was therefore constructed and the foundation was partly made of shingle and partly of sand. Changes in pressure which would be caused by altering part of the sand into shingle were studied. This became very difficult in a hydraulic model and the readings obtained were not consistent. In a model about 15 inches in length, the hydraulic method was found to give variable results. It was suggested that an attempt should be made to examine the problem on the basis of the electrical analogy. While the electrical analogy method had worked very successfully in the case of sound weirs, its applicability to unsound conditions had not been examined. Objections might be raised in applying this method to cavities and this is dealt with later.

A model of Kalabagh weir in ebonite, 15 inches in length, was constructed in the electrical tray. The upstream and downstream ends of the model were made of copper plates as usual. The details of construction of such models have already been given in previous reports. The main difference in the present investigation was that the cavity was represented by a brass block to short circuit the upstream and downstream of the cavity. This is apparently justifiable because in a cavity in the prototype, there is no difference of pressure between the upstream and downstream ends of the cavity.

The first set of readings was taken without the cavity and the next five sets were taken with the cavities in the positions marked in Fig. 194, as 1, 2, 3, 4 and 5. The pressure distributions on the weir corresponding to these respective positions are shown in the figure.

The main conclusions which can be drawn from these curves are as follows :—

(a) The magnitude of the pressure changes brought about by a cavity are small and confined mostly to the neighbourhood of the cavity. For instance, the model of the Kalabagh weir was 15" and the cavity is 2". This represents a cavity which is 2/15 of the length of the weir and yet the difference of potential is only about 8 per cent. It will require very great care and accuracy to detect this difference in a prototype by pressure pipes. It is even doubtful whether pressure pipes can be relied on to indicate changes of this order.

(b) If a cavity is nearer the upstream or downstream end of the weir, the change of pressure caused by it is greater than that which would occur if a similar cavity is situated in the centre.

(c) A cavity increases the pressure gradient on both the upstream and downstream sides of it. Thus a cavity in the neighbourhood of the downstream sheet pile increases the average percentage gradient to 0·57 in its vicinity on the downstream side and to 0·48 on the upstream side. The original percentage gradient without the cavity was about 0·4.

Though, as stated before, the electrical analogy method was found to be true for investigations on sound weirs, its application to weirs with cavities has not been proved. It has been suggested that in the case of a cavity, the potential distribution and pressure distribution cannot be taken to be identical. The point can be settled by an experiment on hydraulic and the corresponding electrical models.

Experiments on Cavities with Hydraulic and Electrical Models—For this work two investigations were required, the first was on the hydraulic model and the second on the electrical model. The experiments on the hydraulic model will be described first.

The model consisted of a tank 125 cm long, 75 cm high and 20 cm broad. A sketch of the tank used for the investigation is shown in Fig. 195. The floor of the model was made of glass 1·5 cm thick 30 cm long and 20 cm broad. To insert pressure pipes, holes were bored through the glass plates at intervals of two and a half centimeters. The pipes for observing the pressure distribution were fitted into these holes. In order to keep the upstream and downstream water at desired levels metallic plates were fixed vertically and syphon arrangements were put in.

Before the floor was fixed on the top of the sand, a cavity two inches in diameter and semi-cylindrical in shape was made across the

FIG. 194

KALABAGH WEIR

PRESSURE DISTRIBUTION BY ELECTRICAL METHOD

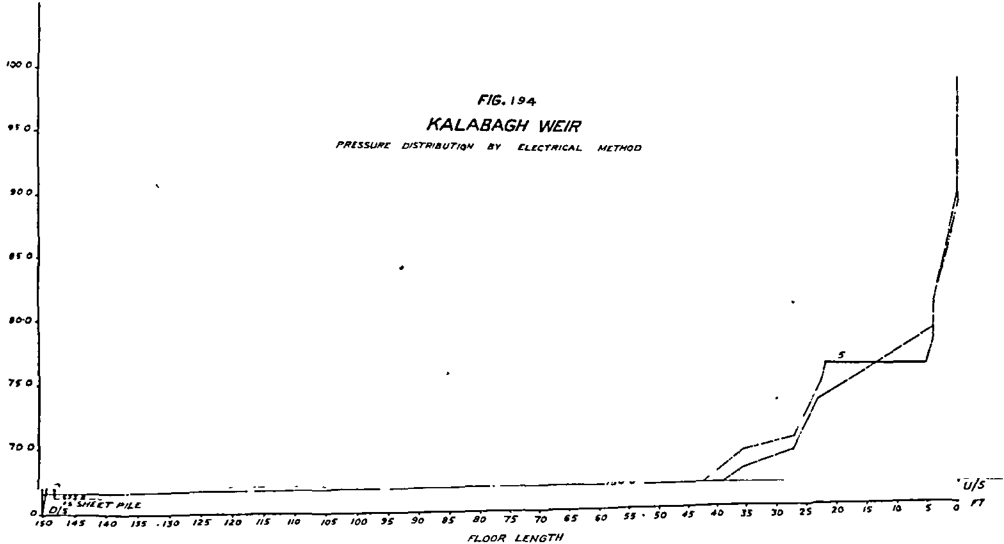


FIG. 195

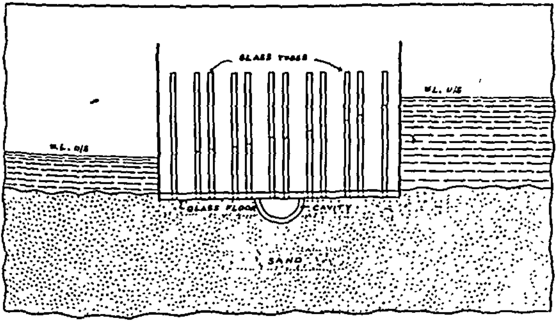


FIG. 196

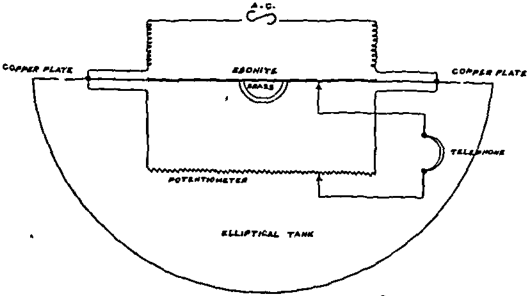
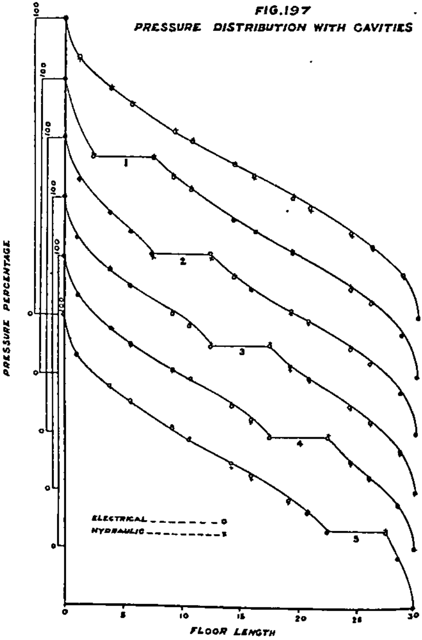


FIG. 197
PRESSURE DISTRIBUTION WITH CAVITIES



bottom. In order to maintain the shape and size of the cavity intact a perforated plate of brass was pressed into the cavity. The perforations were not fine enough to cause any change of pressure at the boundary. The glass floor was fixed to the sand foundation above the cavity.

Five positions of the cavities were investigated and two sets of experiments were carried out, each with a different head. In one set, a head of 7 cm and in the other a head of 4 cm was employed. The results are shown in Tables XV and XVI. Column "a" in the tables gives the actual readings obtained, column "b" gives the percentage hydraulic pressure calculated from the readings and column "c" gives the percentage electrical potential. Having completed this series of experiments in the hydraulic model, it was necessary to try a corresponding model in the electrical tray.

The model in the electrical tray was set up as follows. Corresponding to the glass floor in the hydraulic, an ebonite plate was fixed in the electrical tray. The upstream and downstream levels were thick copper plates. The cavity was represented by a semi-cylindrical piece of brass attached to the bottom of the ebonite plate. The dimensions of the floor and of the cavity in the hydraulic model were thus completely represented in the electrical model without any change of scale. Five positions of the brass cylinder, positions exactly similar to those of the cavities in the hydraulic model were investigated. A sketch of the electrical tray showing one position of the cavity is given in Fig. 196. The alternating current for the investigation was obtained from an oscillator. This has been described in previous reports. The general method of conducting the experiments is clear from the diagram. The potential distribution along the floor with cavities is obtained on a potentiometer. The percentage pressures so obtained and those calculated from the hydraulic model are plotted in Fig. 197.

The corresponding points in the hydraulic and electrical models nearly superpose in the figure. The values agreed within three per cent, and considering the sources of error in the hydraulic model this agreement is excellent.

The investigation thus proves that the pressure distribution under weirs with cavities can be studied electrically and the conclusion obtained from the electrical model are valid for the hydraulic prototype.

Investigations on Pressure Relief in the Bed of the Haveli Main Line—In the neighbourhood of the Haveli Main Line the water-table is higher than the bed level and the bed is lined. When the canal is dry, the pressure from the water-table would tend to blow up the bed. It is, therefore, necessary for a certain head of water to be maintained in the canal to counteract the uplift pressure. In order to avoid this head of water, it was proposed to relieve the excess pressure by means of relief pipes of 2 inch bore and three feet in length

fixed 25 feet apart. The question whether this would be effective was examined in the laboratory.

A model of the canal 1 foot wide, 6 inches deep and about 3 feet long was set up in a tank containing sand. The tank was about $9' \times 4\frac{1}{2}' \times 2\frac{1}{2}'$ in size and was divided into three compartments. The two end compartments were small and contained free water. The middle compartment was filled with sand and the canal was made in the sand. The model of the canal was made of cement. As the cement model of the canal was being built, relief pipes and pipes to measure the pressure were inserted. The relief pipes were arranged in three rows three inches apart and the pressure pipes were midway between them and so also three inches apart. The relief pipes consisted of $\frac{1}{4}$ inch brass pipes with strainers at the bottom and flush with the bed and the pressure pipes were glass tubes, projecting above the floor. These are shown in Fig. 198.

The level of the water could be maintained above the bed at any desired head with the help of a syphon in the tank. As the level of water was raised and it became higher than that of the bed, water began to flow into the canal through the relief pipes.

Table XVII gives the percentage relief of pressure with different heads of water.

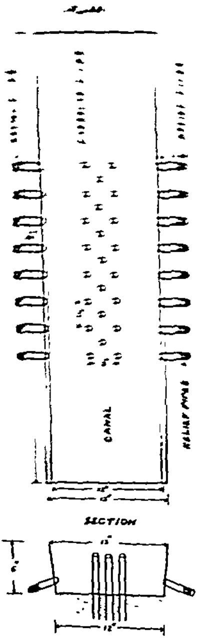
It can be seen from the last column that the pressure relief is negligible. The proposal as it stood was therefore not satisfactory.

A second method was therefore tried. In this method, the strainer of the relief pipes was shrouded with coarse sand graded between 1 mm and 1·5 mm. The shrouding extended to one inch in diameter round the relief pipe. It may be pointed out that the original sand under the bed was 0·03 mm in diameter in the model. The sand specimen obtained from the field under the bed of the canal was still finer. When readings were taken with this shrouding, better results were obtained as could be seen from Table XVIII.

This method while it gave good results was not possible in practice, because it would require the breaking up of the bed at many places in the prototype, removing the bed material and putting in large amount of coarse sand. On these grounds, it had to be rejected.

A third method was therefore tried. In this method, the relief pipes were fixed to the banks of the canal so that the flow lines were intercepted before they reached the bed. The relief pipe was so inclined that the strainer was in level with the bed, but on the outside of it as will be seen from Fig. 198. Secondly a continuous shrouding of coarse sand graded between 1 mm and 1·5 mm was made to fill round the strainers. The shrouding material thus formed a long belt encasing the strainers of the relief pipes. The pipes for pressure observation were in the same position as before. Readings obtained with this arrangement are shown in Table XIX and it will be seen that in this case the relief is about 45 per cent.

[illegible]



CANAL
RELIEF PIPES
SECTION
12"
6"

FIG. 199

APPARATUS FOR PRESSURE DEFICIENCY

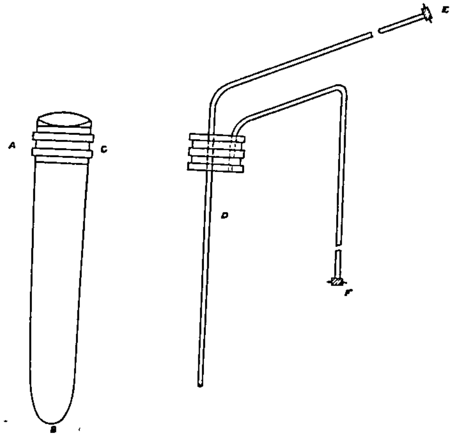
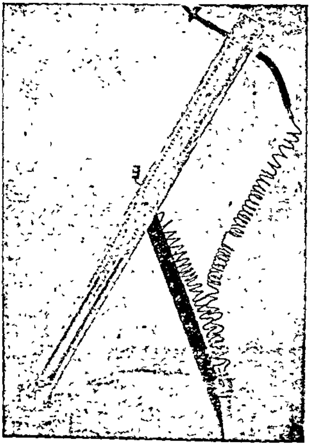


FIG. 200
NEGATIVE PRESSURE APPARATUS



Pressure Deficiency Experiments at Bupra—It was mentioned in last year's report that experiments were in progress in order to determine the pressure deficiency in the field with Roger's apparatus. Experiments at Chianwali were carried out with four sets of this apparatus. As it was difficult to obtain further supplies an alternative apparatus was constructed in the laboratory. A sketch of this is shown in Fig. 199 and a photograph in Fig. 200. ABC is a porous pot about six inches in length and one inch in diameter slightly tapering towards the bottom. A brass ring with screws which has been specially moulded and made to fit A B C was constructed and fitted. D is a metallic screw cap fitted with metallic pipes. The metallic pipes are copper tubes used for petromax lamps and soldered to the brass cap. The tube E goes to the bottom of the porous pot and is connected to the manometer. The tube F is meant to fill the porous pot with water. The metallic cap D has a rubber ring so that when it is screwed on to the porous pot, it fits tightly and the arrangement is water tight. The metallic tubes E and F are long enough so that their ends are out of the soil when embedded in the field. Tight fitting screws close their mouths at the upper ends.

This apparatus is simpler to use than Roger's for the latter is meant for cold climates where water freezes in the apparatus. This difficulty does not occur in the Punjab. This apparatus along with some of Roger's apparatus, have been used to find the pressure deficiency in a field at Bupra.

Bupra is in Gujranwala District and the water-table in the area where these experiments were carried out was about eight feet below N. S. The soil was hard and a mechanical analysis is given in Table XX.

The apparatus were placed at depths of 1½, 3, 4½ and 6 feet below the natural surface. Observations were taken from the middle of March 1941 up to August 1941. Two readings of the pressure deficiency were taken daily. It is not possible to give all these readings in the report; weekly readings are given in Table XXI.

It will be seen from Table XXI that the first apparatus, 1½ feet below natural surface, gave continuously increasing readings of pressure which reached a maximum by the end of June. When the first rain fell on the 1st of July this pressure deficiency dropped from 57·8 to 4 cm of mercury. It is therefore clear that the effect of rain is felt at this depth. The second apparatus which is three feet below natural surface showed a fall from 64 to 10 cm of mercury. At this depth also therefore the rain has increased the moisture content considerably. The apparatus at four and a half feet depth fell from 15·3 cm to 9·5 cm only, still showing that the rain affects the moisture content to this depth. The apparatus at six feet below the natural surface did not show any change after rain. Its original reading of 10 cm continued without decrease. This instrument at the sixth foot remained constant throughout the whole period of investigation.

The moisture percentage at this depth in this soil is not affected by heat and rain on the natural surface. This instrument is a foot and a half above the water level recorded by a pipe situated near it. It appears that moisture content at this depth is largely controlled by the water-table. Further experiments are now being carried out near Sambrial.

Testing of Samples for the High Dam Circle—Twelve samples of stone were investigated for their porosity in connection with the construction of dams. The High Dam Circle submitted specimens to determine whether there would be any percolation through them under a pressure of 40 pounds per square inch.

This experiment was carried out as follows. The stone pieces were cut into a T shape as shown in Fig. 201. These were about two inches in length and two inches in cross section at the broadest part. They were fitted to form a plug of a specially constructed chamber, so that the side A is under a pressure of 40 pounds per square inch and side B is at atmospheric pressure. The pressure was applied with a hydraulic pump. If any percolation took place from side A to side B, the hydraulic pressure would fall and this could be read on a pressure gauge attached to the hydraulic pump. Twelve samples were investigated in this manner and all of them stood the pressure showing no percolation.

It was then thought advisable to obtain a qualitative idea if an ordinary brick could stand any hydraulic pressure. When similar experiments were performed with bricks it was found that the pressure fell almost instantaneously. This latter experiment was carried out as a check on the method.

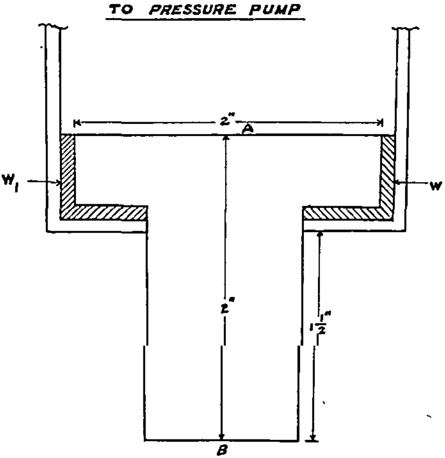
Silt analysis

One thousand four hundred and seventy silts were analysed during the year under report. Out of these one thousand and fifty were from the regular sites and the rest were in connection with other investigations.

Evaporation from soils—Evaporation of water from soils is a [illegible] in various countries and is one of great [illegible] ependence of evaporation on the grade [illegible] en, and it has been concluded that the [illegible] constant at high moisture contents. Fisher has attributed this to the fact that actually water is evaporating from a free water surface.

Certain experiments which were carried out in this laboratory with different grades had also shown that evaporation was independent of the grade at high moisture contents. These and similar experiments by other workers were carried out using very thin layers of soil. The effect of the depth of the column of sand has not been investigated in detail. In the Punjab where the temperature varies considerably between winter and summer and where the climate is dry, conditions are very different from those in which the experiments have been

FIG. 201



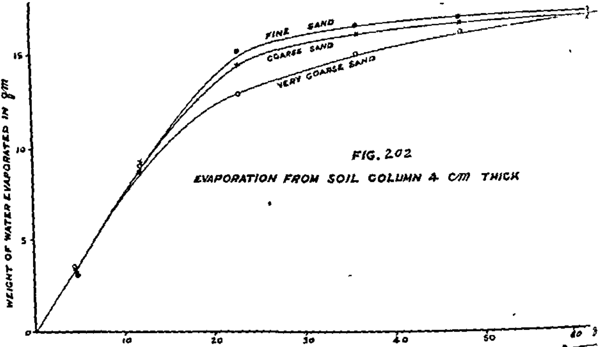
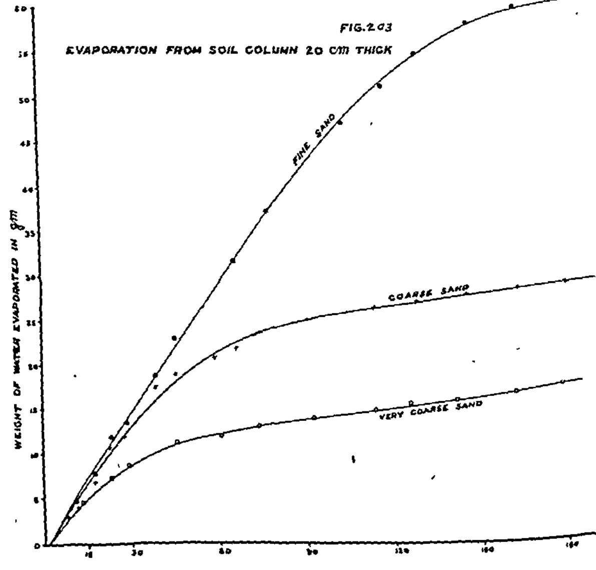


FIG. 202

EVAPORATION FROM SOIL COLUMN 4 cm THICK

FIG. 203

EVAPORATION FROM SOIL COLUMN 20 cm THICK



carried out by workers abroad. It was thought desirable to re-investigate the problem.

Two series of experiments were carried out with very coarse, coarse and fine sand; the depth of the layer of sand in the first series was 4 cm, and in the second series was 20 cm. The very coarse sand was about 2 mm. in mean diameter, the coarse 0·8 mm in mean diameter, and the fine sand was about 0·2 mm in mean diameter. The soils were packed under water contained in metallic tubes 1·5 cm in diameter, the heights being 4 and 20 cm as referred to before. Evaporation was allowed to take place in the open and was measured by weighing the tubes daily. The observations were continued till it was found that the rate of evaporation had become negligible.

In the 4 cm layer the evaporation became negligible after a week; and in the layer 20 cm deep it became negligible in about three weeks.

In order to compare the evaporation with that from a free water surface a tube of the same bore was filled with water and water was allowed to evaporate under similar conditions. The water which evaporated daily was replaced. The results obtained were not in conformity with the opinion that evaporation is independent of the grade of material. The rate of evaporation depends on the grade and on the depth of the column of sand under experiment and not on one factor alone.

The results are shown in Figs. 202 and 203. To represent the results the abscissa is taken as the accumulated daily evaporation from the water surface and the ordinate as the weight of water evaporated from the respective sand columns. This gives a better representation of the readings than a time axis for the abscissa for it eliminates the changes in the atmospheric conditions.

Referring to Fig. 202 it will be seen that when the column of sand is 4 cm in length the difference of evaporation between the various grades is very small. Conclusions from such experiments lead one to assume that evaporation is independent of grade. Under this condition it is true.

Referring to Fig. 203 it will be seen that when the evaporating column of sand is 20 cm, the evaporation from finer grades of sand is greater than that in the coarser material. For instance, when 100 gm. of water had evaporated from the water surface, 14·7, 26·1 and 46·7 gm of water had evaporated from very coarse, coarse and fine sands, respectively. This is contrary to the observations referred to in the last paragraph and to those in old literature on the subject. One worker, however, named Esser, has in 1884 shown that if evaporation from a 1,000 sq. cm. surface from soil fractions less than 0·071 mm in particle size is taken as 100, that from fractions between 0·25 to 0·5 mm is 81·1 and that from fractions between 1 to 2 mm is 22·2. He, however, does not state the depth of the columns of sand used.

These experiments will be continued and it is hoped then that it will be possible to review the whole subject and its bearing on problems of irrigation and evaporation.

TABLE XV

COMPARISON OF PRESSURE DISTRIBUTION WITH CAVITIES IN THE ELECTRICAL AND HYDRAULIC MODELS.

	Without Cavity			I			II			III			IV			V		
	a	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*
Upstream level ..	7·08	100·0	100·0	7·12	100·0	100·0	7·08	100·0	100·0	7·28	100·0	100·0	7·07	100·0	100·0	6·88	100·0	100·0
1	6·15	86·8	87·0	5·98	83·0	84·3	6·08	85·0	86·1	6·29	86·4	86·3	6·07	85·0	86·5	5·94	86·3	86·7
2	5·45	77·0	76·8	5·26	73·0	73·5	5·29	74·7	74·7	5·51	75·7	75·3	5·35	75·7	75·8	5·20	75·6	75·8
3	5·08	71·7	71·3	5·26	73·9	73·5	4·93	69·6	68·9	5·10	70·1	70·3	4·94	69·8	70·8	4·81	69·9	71·0
4	4·45	62·8	62·4	4·78	67·2	66·5	4·26	60·2	61·0	4·47	61·4	60·8	4·35	61·6	61·6	4·26	61·9	61·5
5	4·23	59·7	59·2	4·49	63·1	62·5	4·26	60·2	61·0	4·20	57·7	57·0	4·14	58·6	58·0	4·01	58·3	57·5
6	3·65	51·6	51·3	3·77	52·0	53·5	3·63	54·1	53·7	3·70	50·8	50·0	3·59	50·8	49·7	3·32	48·2	49·5
7	3·36	47·4	47·6	3·51	49·3	49·5	3·48	49·2	49·3	3·70	50·8	50·0	3·18	45·0	45·5	3·04	44·2	45·3
8	2·92	41·2	40·7	3·03	42·5	42·3	2·90	41·0	42·0	3·09	42·4	43·1	2·79	39·5	39·0	2·50	36·4	37·4
9	2·63	37·1	37·5	2·74	38·5	38·8	2·71	38·2	38·5	2·83	38·8	39·4	2·79	39·5	39·0	2·33	33·8	33·5
10	2·01	28·4	28·6	2·9	30·7	29·8	2·12	30·0	29·5	2·12	29·1	29·8	2·16	30·5	31·3	1·84	26·8	26·5
11	1·64	23·2	23·8	1·64	25·0	25·2	1·74	24·6	24·3	1·83	25·1	24·8	1·82	25·8	25·5	1·84	26·8	26·5
12	1·01	14·3	14·5	1·07	15·0	14·6	1·07	15·1	14·7	1·08	14·8	15·2	1·11	15·7	16·4	1·17	17·0	17·1
Downstream level ..	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0

TABLE XVI

COMPARISON OF PRESSURE DISTRIBUTION WITH CAVITIES IN THE ELECTRICAL AND HYDRAULIC MODELS.

	Without Cavity			I			II			III			IV			V		
	a	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*	*a*	*b*	*c*
Upstream Level	4·39	100·0	100·0	4·16	100·0	100·0	4·08	100·0	100·0	4·26	100·0	100·0	4·41	100·0	100·0	4·31	100·0	100·0
1	3·80	86·6	87·0	3·49	84·0	84·3	3·52	86·2	86·1	3·69	86·6	86·5	4·21	85·7	86·5	3·71	86·1	86·7
2	3·38	77·0	76·8	3·09	73·9	73·5	3·05	74·7	74·7	3·23	75·8	75·3	3·79	75·7	75·8	3·27	75·7	75·8
3	3·14	71·6	71·3	3·07	73·9	73·5	2·84	69·5	68·9	2·99	70·1	70·3	2·91	69·9	70·8	3·01	69·9	71·0
4	2·75	62·6	62·4	2·79	67·1	66·5	2·45	60·0	61·0	2·63	61·8	60·8	2·61	61·9	61·6	2·67	61·9	71·5
5	2·62	59·7	59·2	2·63	63·1	62·5	2·45	60·0	61·0	2·44	57·2	57·0	2·47	58·6	58·6	2·50	58·0	57·5
6	2·23	51·6	51·3	2·21	53·1	58·5	2·20	53·9	53·7	2·16	50·8	50·0	2·14	50·8	49·7	2·09	48·5	49·5
[illegible]	2·10	47·9	47·6	2·05	49·3	49·5	2·01	49·2	49·3	2·16	50·8	50·0	1·89	45·0	45·5	1·91	44·4	45·3
[illegible]	1·81	41·2	40·7	1·77	41·6	42·3	1·62	41·2	42·0	1·82	42·6	43·1	1·67	39·6	39·0	1·59	36·8	37·4
[illegible]	1·66	37·8	37·5	1·60	38·5	38·8	1·56	35·2	38·5	1·65	38·8	39·4	1·67	39·0	39·0	1·40	33·8	33·5
[illegible]	1·24	28·4	28·6	1·26	30·2	29·8	1·23	30·2	29·5	1·24	29·1	29·8	1·30	30·8	31·3	1·16	26·9	26·5
[illegible]	[illegible]	[illegible]	25·8	1·08	25·9	25·2	1·01	24·7	24·3	1·07	25·2	24·8	1·09	25·8	25·5	1·16	26·9	26·5
[illegible]	[illegible]	[illegible]	14·5	[illegible]	15·0	14·6	0·64	15·0	14·7	0·63	14·8	15·2	0·66	15·7	15·4	0·75	17·4	17·1
[illegible]	[illegible]	[illegible]	[illegible]	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0	0·00	0·0	0·0

TABLE XVII

PRESSURE RELIEF

Obs. No.	Level of water-table.	Level of water in the canal	Level of water in the pressure pipes	Difference of level between surface of water in the canal and water-table.	Difference in level due to relief pipes.	% pressure relieved.
	cm.	cm.	cm.	cm.	cm.	
1 ..	12·7	2·54	12·2	10·16	·5	4·9
2 ..	10·16	2·54	9 81	7·62	·36	4·7
3 ..	7·62	2·54	7·40	5·08	·22	4·3
4 ..	5·08	2·54	4 97	2·54	·11	4·3

TABLE XVIII

PRESSURE RELIEF

Obs. No.	Level of water-table	Level of water in the canal	Level of water in the pressure pipes	Difference of level between surface of water in the canal and water-table	Difference in pressure due to relief pipes	% pressure relieved
	cm.	cm.	cm.	cm.	cm.	
1 ..	12·7	2·54	8·25	10·16	4·45	43·8
2 ..	10·16	2·54	7·20	7·62	2·96	38·8
3 ..	7·62	2·54	5·42	5·08	2·20	43·3
4 ..	5·08	2·54	3·80	2·54	1·28	42·8

TABLE XIX

PRESSURE RELIEF

Obs. No.	Level of water-table.	Level of water in the canal.	Level of water in pressure pipes	Difference of level between surface of water in the canal and water-table.	Difference of pressure, due to relief pipes,	% pressure relieved.
	cm.	cm.	cm.	cm.	cm.	
1 ..	12·70	2·54	7·80	10·16	4·90	48·2
2 ..	10·16	2·54	6·80	7·62	3·36	45·6
3 ..	7·62	2·54	5·42	5·08	2·20	43·2

TABLE XX

MECHANICAL ANALYSIS

Depth below N. S. of soil at Rupra	% clay	% Fine Silt	% Silt
1·5 feet	22·40	21·38	13·03
3 „	26·65	24·85	14·75
4·5 „	32·50	31·03	10·40
6 „	17·38	15·20	8·38

TABLE XVII

PRESSURE RELIEF

Obs. No.	Level of water-table.	Level of water in the canal	Level of water in the pressure pipes	Difference of level between surface of water in the canal and water-table.	Difference in level due to relief pipes.	% pressure relieved.
	cm.	cm.	cm.	cm.	cm.	
1 ..	12·7	2·54	12·2	10·16	·5	4·9
2 ..	10·16	2·54	9·81	7·62	·36	4·7
3 ..	7·62	2·54	7 40	5·08	·22	4·3
4 ..	5·08	2·54	4·97	2·54	·11	4 3

TABLE XVIII

PRESSURE RELIEF

Obs. No.	Level of water-table	Level of water in the canal	Level of water in the pressure pipes	Difference of level between surface of water in the canal and water-table	Difference in pressure due to relief pipes	% pressure relieved
	cm.	cm.	cm.	cm.	cm.	
1 ..	12·7	2·54	8·25	10·16	4·45	43·8
2 ..	10·16	2·54	7·20	7·62	2·96	38·8
3 ..	7·62	2·54	5·42	5·08	2·20	43·3
4 ..	5·08	2·54	3·80	2·54	1·28	42·8

MATHEMATICAL SECTION

Regime Channels—The channels given in Table XXII have been kept under observation during 1941-42. Observations on some of these channels were discontinued during the year as, due to staking and bushing, considerable changes took place in their regime.

Analysis of the data from observations at regime sites led to the following relationships :—

$$S \times 10^3 = 2 \cdot 09 \frac{m^{\cdot 86}}{Q^{\cdot 21}} \quad .. \quad .. \quad .. \quad (1)$$

$$P_w = 2 \cdot 8 \sqrt{Q} \quad .. \quad .. \quad .. \quad (2)$$

$$R = \cdot 47\, Q^{\frac{1}{3}} \quad .. \quad .. \quad .. \quad (3),$$

where Q is the discharge in cusecs, R the hydraulic mean depth, P_w the wetted perimeter, S the slope per thousand and m the average diameter of the bed silt in mm. Attempts have been made to introduce the quantity of silt rolling on the bed into equation (1) but this has not been successful. Experiments conducted in this connection will be described later. For the purpose of design, whenever the diameter of the bed silt is known, the slope required can be calculated with a fair degree of precision. Complaints have been received about the silting of channels over the design section. On analysis of the bed silt obtained from the site it was found that the channel had silted and assumed a slope which Eq. (1) should give for the bed silt present. Recently a survey of the main line and branches of the Lower Chenab Canal system has afforded an opportunity of studying this problem more carefully. It has been found that in those channels where the bed did not show any sign of silt movement the actual water-surface slope agreed closely with the calculated slope from Eq. (1). In other cases, such as the Lower Gugera Branch or the Burala Branch, which show unmistakable signs of bed silt movement in certain reaches, the slope is very much in excess of that required by Eq. (1). This equation therefore gives the slope or the full supply level required. The problem of design now resolves itself into finding the following :—

(1) The water-surface slope or the F. S. L.—This will be done with the help of Eq. (1).

(2) The cross-sectional dimensions of the channel.

The two other relations given above determine the size and shape of the channel. One of these is Lacey's P-Q relationship with a slightly altered constant, viz.—

$$P_w = 2 \cdot 8\, Q^{\frac{1}{2}} = C\, Q^{\frac{1}{2}} \quad .. \quad .. \quad .. \quad (2)$$

Though the relation between P and Q has been found to be almost perfect from the statistical point of view, yet great difficulties

TABLE XXI

READINGS OF NEGATIVE PRESSURE

Date		1½ feet	3 feet	4½ feet	6 feet	Water-table
18th March 1941	..	6·1	20·1	15·2	10·1	7′—6·5″+4″
Ditto	..	6·2	19·8	15·2	10·1	7′—6·9″
25th March 1941	..	14·5	12·4	19·2	10·2	7′—9″
Ditto	..	13·5	13·7	17·5	10·2	7′—9·3″
1st April 1941	.	18·7	13·2	13·7	10·0	7′—11″
Ditto	.	19·0	12·6	13·5	10·0	7′—11″
8th April 1941	.	21·0	14·5	12·6	10·1	7′—9·6″
Ditto	.	20·9	13·5	12·4	10·1	7′—10·5″
15th April 1941		28·2	17·0	12 5	10·5	7′—9·2″
Ditto	.	28·4	15·3	12·2	10·5	7′—9″
22nd April 1941		37·4	22·2	10·0	10·4	8′—3·7″
Ditto		37 9	21·4	10·1	10·3	8′—4″
29th April 1941	..	42 0	29·0	10·3	10·4	8′—6·6″
Ditto	..	42·4	29 0	10·3	10·0	8′—5″
6th May 1941	.	48·6	41·2	13·0	10·4	8′—1·6″
Ditto	.	48·8	42·0	13·0	10·4	8′—2·4″
13th May 1941	.	53·5	50·5	13·0	10 0	8′—3·5″
Ditto	.	53·9	50·5	13·0	10·0	8′—3·5″
20th May 1941		54·3	55·0	13·8	10·0	8′—2·0″
Ditto	..	54·4	55·5	13·8	10·0	8′—2″
27th May 1941	.	56·8	59·5	14·0	10·2	8′ 1″
		Refilled				
5th June 1941	.	44·0	62·6	16·0	10·8	8′—6·1″
Ditto	..	44·0	62 1	16·1	10·8	8′—6·3″
12th June 1941		53·6	63 5	16·0	11 0	8′—5·5″
19th June 1941	.	58·0	64·1	9·3	9 7	8′—3 8″
Ditto	.	58·0	64·0	9·5	10·0	8′—3·8″
24th June 1941	..	57·8	63 6	15·1	10·0	8′—2″
Ditto	..	57·5	60·3	15·2	10·0	8′—2″
1st July 1941		57·8	64·0	15·3	10·0	8′—1″ Rainfall—On 3rd July 1941=2 87″ On 6th July 1941=·97″ On 9th July 1941=·33″
8th July 1941	..	4·0	10·0	9·5	10·15	7′—6·5″
Ditto	..	4·0	12·0	9·5	10·15	7′—6·5″
15th July 1941	..	3·9	32·5	15·0	10·15	7′—7·5″
Ditto	..	3·9	33·4	15·1	11·5	7′—7·5″ Rainfall—On 17th July 1941=·48″ On 18th July 1941=·25″
22nd July 1941	..	4·0	50·0	22·7	12·6	7′—5·4″
Ditto	..	4·0	50·3	22·8	12·6	7′—5·3″
		Refilled				
29th July 1941	..	49·5	60·0	27·3	11·8	7′—3·7″ Rainfall On 29th July 1941=·21″
4th August 1941	..	40·0	63·5	33·5	11·5	7′—4·5″

MATHEMATICAL SECTION

Regime Channels—The channels given in Table XXII have been kept under observation during 1941-42. Observations on some of these channels were discontinued during the year as, due to staking and bushing, considerable changes took place in their regime.

Analysis of the data from observations at regime sites led to the following relationships:—

$$S \times 10^3 = 2 \cdot 09 \frac{m^{.86}}{Q^{.21}} \quad \ldots \quad \ldots \quad \ldots \quad (1)$$

$$P_w = 2 \cdot 8 \sqrt{Q} \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

$$R = \cdot 47\, Q^{\frac{1}{3}} \quad \ldots \quad \ldots \quad \ldots \quad (3),$$

where Q is the discharge in cusecs, R the hydraulic mean depth, P_w the wetted perimeter, S the slope per thousand and m the average diameter of the bed silt in mm. Attempts have been made to introduce the quantity of silt rolling on the bed into equation (1) but this has not been successful. Experiments conducted in this connection will be described later. For the purpose of design, whenever the diameter of the bed silt is known, the slope required can be calculated with a fair degree of precision. Complaints have been received about the silting of channels over the design section. On analysis of the bed silt obtained from the site it was found that the channel had silted and assumed a slope which Eq. (1) should give for the bed silt present. Recently a survey of the main line and branches of the Lower Chenab Canal system has afforded an opportunity of studying this problem more carefully. It has been found that in those channels where the bed did not show any sign of silt movement the actual water-surface slope agreed closely with the calculated slope from Eq. (1). In other cases, such as the Lower Gugera Branch or the Burala Branch, which show unmistakable signs of bed silt movement in certain reaches, the slope is very much in excess of that required by Eq. (1). This equation therefore gives the slope or the full supply level required. The problem of design now resolves itself into finding the following :—

(1) The water-surface slope or the F. S. L.—This will be done with the help of Eq. (1).

(2) The cross-sectional dimensions of the channel.

The two other relations given above determine the size and shape of the channel. One of these is Lacey's P-Q relationship with a slightly altered constant, viz.—

$$P_w = 2 \cdot 8\, Q^{\frac{1}{2}} = C\, Q^{\frac{1}{2}} \quad \ldots \quad \ldots \quad \ldots \quad (2)$$

Though the relation between P and Q has been found to be almost perfect from the statistical point of view, yet great difficulties

have been experienced in fixing the constant 2·8. It varies considerably from channel to channel and also on the same channel. Mr. Lacey in his note to the Research Officers' Meeting of the Central Board of Irrigation (July, 1941) has suggested the use of the water-surface width W instead of the wetted perimeter P in the following form :—

$$P = E.\ K.\ Q^{\frac{1}{2}}$$

where $E = \dfrac{P}{W}$ reducing the above to

$$W = K\ Q^{\frac{1}{2}},$$

the old Schocklitsch relation for rivers. Calculations of the Punjab regime site data have shown that this suggestion does not lead to any improvement. (Table LIV).

The other relation between R and Q suffers from the same defect as the relation between P and Q. The variation in the constant is almost of the same order as that in Eq. (2).

This had led to the examination of the data of the Punjab regime sites from a different point of view. The values of $P/\sqrt{Q}=C_1$ and $R/Q^{\frac{1}{3}}=C_2$ have been calculated for all the sites and their percentage variation from the constants given in Equations (2) and (3) have been tabulated in Table XXIII. A careful study of the table shows that in almost all cases where the variation of the constant in Eq. (2) has been on the positive side that in Eq. (3) has been on the negative side. This means that where the channel has become wider than is required by Eq. (2) the depth has become shallower than required by Eq. (3). This observation is very significant.

A survey of all these regime sites was undertaken during the Winter Closure of 1940-41 in connection with Lacey's "Theory of Shock". The following observations were carried out at all the regime sites :—

(*i*) Cross Section of the channel at the observation site and at every 100 feet upstream and downstream of the site for a distance of 1,000 feet on either side—R.Ls. of the natural surface were taken at the same time.

(*ii*) Soil samples from both the berms at each cross-section.

(*iii*) Photographs of the berms and the bed at representative points of the canal over the site.

While carrying out this survey it was noticed that the width of the canal appeared to depend on the nature of the soil comprising the berm. In consequence the variation of the constant in Eq. (2) was to some extent determined by the character of the berm soil. If the soil contained salt, the berm did not stand the action of flowing

water and fell in, thus widening the channel. When the channel had widened, the discharge being constant, it tended to silt, keeping the waterway constant. If the waterway altered it can be presumed that the water surface slope would adjust itself to accommodate the altered waterway. As has been pointed out, a reference to Table XXIII will confirm this. When the percentage variation of C_1 is positive that of C_2 is negative, that is when the wetted perimeter P is increased the depth is reduced and vice versa. This occurs in almost all the cases excepting Nos. 4.15, 25, 31, 34, 38, 39 and 41. In consequence, it was to be expected that the waterway A=P. R would be highly correlated with the discharge Q. This is in fact the case. The correlation between A and Q is 99·91 per cent and the variation in the constant C_3 in

$$\frac{A}{Q^{\cdot 85}} = C_3 \qquad .. \qquad .. \qquad .. \qquad (4)$$

is very limited. Only in those cases where C_1 and C_2 deviate the same way from the mean value, are the values of C_3 also appreciably different from the average. It is easy to understand that when P and R both increase A will also increase, but it is difficult to explain how the channel passing the same discharge through the increased waterway will maintain equilibrium unless the slope changes. Of the cases quoted above only the cases of Nasrana Distributary, Southern Branch, Lower Jhelum Canal, Main Line R. D. 93,400 and 115,000 of Lower Bari Doab Canal and Dhuniwala Distributary require special consideration. Even in these cases, the actual water-surface slopes do not differ greatly from the slopes calculated from Eq. (1). When the slope is introduced into equation (4) the correlation is not improved and the slope comes in with a very low power of S, viz—·02. The effect of slope can therefore be considered as negligible. Thus for the Punjab regime sites the following equation holds—

$$A=1.145\ Q^{\cdot 85} \qquad .. \qquad .. \qquad .. \qquad (4)$$

This will replace Eqs. (2) and (3). In this case the correlation is as high, if not higher, than in Eqs. (2) and (3) and the variation of the constant 1·145 is much reduced.

This equation though fundamentally correct does not lead very far in designing channels. A study of the existing literature on canal engineering shows that there are two opinions regarding the calculation of the bed width and depth. The first opinion represented by Kennedy, Lindley, Woods, preferred to calculate bed width and depth directly, whereas the second represented by Lacey, take P and R as fundamental hydraulic constants and calculate bed width and depth on the assumption of a trapezoidal or elliptical section. In the Memoir of the Irrigation Research Institute (Vol. II, No. 23) the Lacey method was

followed and B and D were calculated from P and R. This had the disadvantage, as pointed out above, that due to the considerable divergence of the calculated values of P and R from the actual values, the derived values of B and D were unsatisfactory for the purpose of design. The present Eq. No. (4) will, however, give the total waterway necessary for passing the required discharge. But for the purpose of design this is not enough, one of the linear dimensions of the section must also be known, either the depth or the water-surface width. The width has the same disadvantages as the wetted perimeter P and, consequently, the depth was chosen for the purpose of analysis.

A practical difficulty was encountered at the very outset of this investigation; the depth in any cross section of a channel is not uniform. To get over this difficulty, depth D was defined as the mean central depth neglecting the slope segment. It must be admitted that this definition of depth D is purely arbitrary and cannot be expected to have any fundamental significance. It has the advantage of convenience and the high correlation obtained indicates that it has some significance.

This depth has been correlated with the discharges from the regime channels of the Punjab and gives a very high order of correlation which is further improved by the introduction of slope which comes in with a high power. The final equation assumes the following form :—

$$D = 0 \cdot 39 \frac{Q^{\cdot 85}}{S^{0 \cdot 37}} \qquad \ldots \qquad \ldots \qquad \ldots \qquad (5)$$

This has a correlation co-efficient of 99·95 per cent. In Table XXIV the calculated value of D and the divergence of the calculated values from the actual values have been tabulated. This shows that the maximum divergence is of the order of 13·9 per cent.

Having obtained D from Eq. (5) and the waterway from Eq. (4) it is possible now to design the channel. Eq. (1) gives the water-surface slope or the full supply level shown by line XY in Fig. given below—

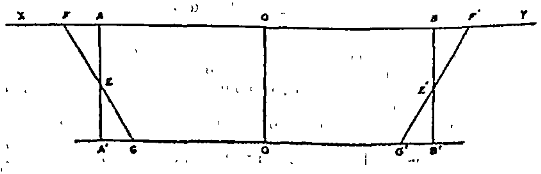
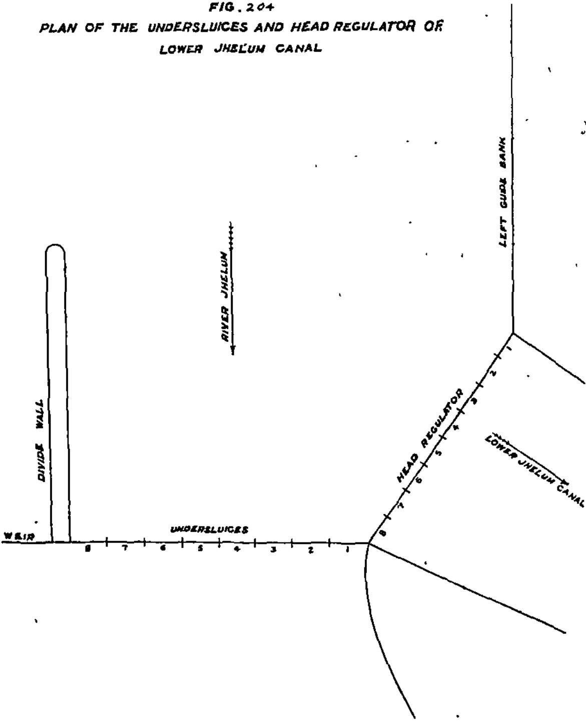


FIG. 204

PLAN OF THE UNDERSLUICES AND HEAD REGULATOR OF LOWER JHELUM CANAL



Let OC be the centre line of the canal section, OC being the depth D obtained from Eq.(4), and AB a length which, with D as the other arm, forms a rectangle whose area is equal to the area A obtained from Eq. (4).

The engineer is now provided with all the information that equations Nos. 1, 4 and 5 can supply. He will have to apply his local knowledge to design the exact section. This he can do in the following way. Suppose he knows that the soil through which the canal is to be taken consists of very hard clay and the berm can stand at an angle even steeper than $\frac{1}{2}:1$. Through the middle points E and E′ of the verticals AA′ and BB′ of the rectangle he can draw the line FEG and F′E′G′ at an angle to which the berms can stand. If the berm soils are of *kalar* he will have to make these lines slope at a much flatter angle. If it is desired to have berms formed from fine silt obtained from the flowing water, the engineer will excavate the channel to a wider section and bush so that the finally formed berms take the slope which is natural to such a formation. The final canal section is given by FEG G′ E′ F′ which has got the same full supply level EF′ as required by Eq. (1), the same depth OC as required by Eq. (5) and the same waterway as required by Eq. (4). The water-surface width will be determined by the local conditions which the engineer will be able to fix knowing the nature of the soil through which he is going to excavate the channel or through which the channel is ultimately going to flow.

Silt Observations—Special silt observations were carried out at the following headworks specially with a view to find the effect of regulation on the silt entry into the canal :—

(*i*) Rasul Headworks on the River Jhelum.

(*ii*) Trimmu Headworks on the River Chenab.

(*iii*) Sidhnai Headworks on the River Ravi.

Rasul Headworks—This headworks has no silt excluding device at the head of the canal, Lower Jhelum Canal. Rigid " Still Pond " system of regulation is practised at the head. The head regulator has 8 bays with adjustable cills and the undersluice has 6 bays. They are numbered as shown in the plan attached (Fig. 204). The cills of the different bays of the canal regulator were adjusted to certain definite R. L's. and then silt samples were collected from the following points and analysed for coarse silt content only ($>\cdot 2$ mm) :—

(*a*) *River upstream along line 13-14.*—This section is in the left area of the river at a distance of 500 feet from the upstream nose of the divide wall. Bottle samples were taken at ·6 depth of 5 verticals. These samples were mixed together and analysed for coarse silt content.

(*b*) *Pocket along R. D. 700*—Silt entering the pocket was obtained by bottle sampling at six verticals along R. D. 700 and analysed, after mixing, for the coarse component.

(c) *Boil of the regulator bays.*—Samples were obtained from the boils and analysed separately for each bay.

Besides these the pond level and the depth of silt in the pocket was also observed between R. L. 100 and 600. These are all given in Table XXV. From these the following conclusions can be drawn :—

(1) Bay No. 8 of the Canal Regulator draws the minimum quantity of silt, whatever the position and arrangement of the cills.

(2) But there is nothing to choose between Bays No. 1 and 7 so far as silt drawing capacity is concerned. Out of the 8 cases given in the Table, Bay No. 7 draws more silt than Bay No. 1 in 3 cases, less in 3 cases and equal in 2 cases.

(3) Table XXV further … on 31st July 1941 (9·15 a.m. and 1 … ore or less the same, the ratio of th … ket silt is of the order 1 : 4 or 1 : 5 whereas on 22nd July 1941 when the cill levels differ by 1·4 feet this ratio remains the same.

These observations do not show that the adjustment of the cill levels of the canal regulator bays has any appreciable effects on the silt draw into the different bays of the canal.

In view of the fact that similar experiments at other headworks have given different results it is proposed to carry out the following observations at Rasul next summer as soon as there is sufficient silt in the river water. Water samples will be taken simultaneously by four bottle samplers before the bays of the regulator in the following arrangement. Samples from … on- secutively at a time and eacl … of 2 hours throughout the day. … —

(1) all the cills at the same level,

(2) the cills rising in a wedge to the right by 0·1′ from one cill to the next.

The cill level for Bay No. 8 should remain the same in both the settings.

In taking samples from the boil the bays should be arranged in the following way :—

(1) Bay No. 1, 3, 5, 7.

(2) Bay No. 2, 4, 6, 8.

Corresponding to (1) there will be 2 settings of the cills and similarly for (2).

It will be necessary to have the following observations at the same time :—

(1) Pond level.

(2) Depth of silt in the pocket.

FIG 205

TRIMMU HEADWORKS

SILT SAMPLING IN THE LEFT POCKET

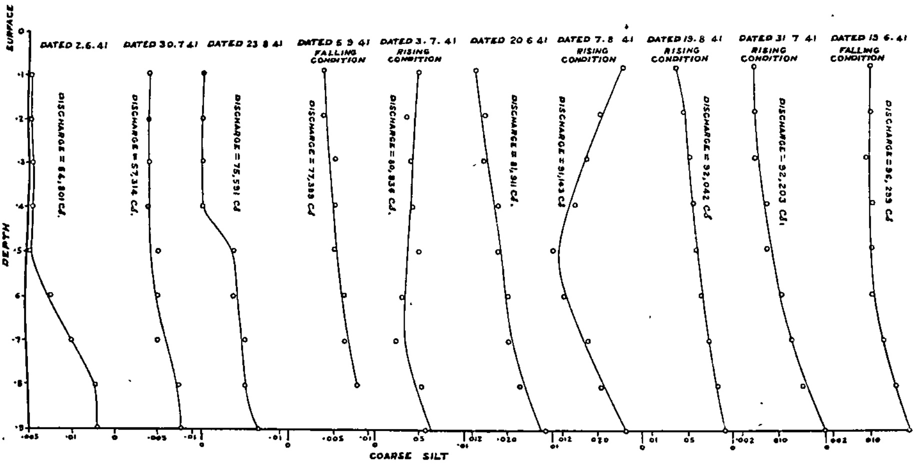


FIG. 205

TRIMMU HEADWORKS

SILT SAMPLING IN THE LEFT POCKET

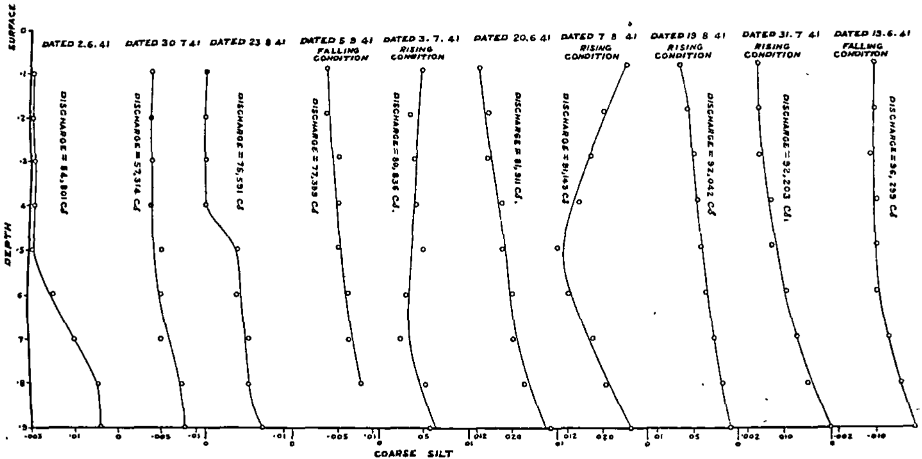
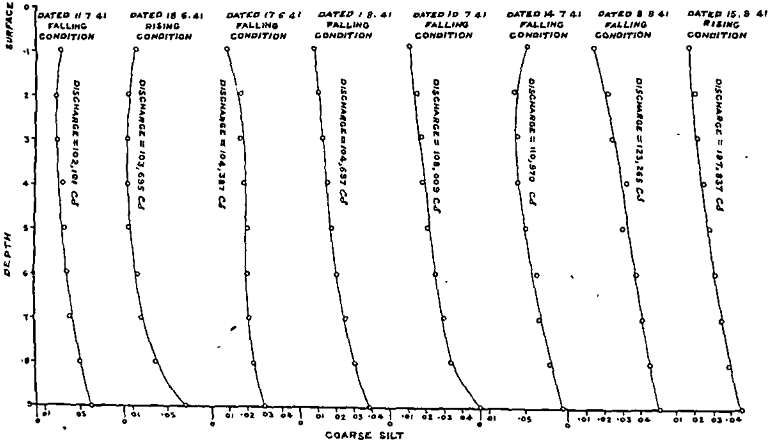


FIG. 206

TRIMMU HEADWORKS

SILT SAMPLING IN THE LEFT POCKET



(3) Slope of the river towards the pocket.

(4) Discharge in the left arm of the river.

Trimmu Headworks—In view of the experience gained while carrying out the silt observations at this headworks during the summer of 1940 the programme of work had been modified. The programme followed in 1940-41 was given in the Annual Report of the Institute for 1940-41. The following is the modified or additional programme followed during 1941 :—

Left Pocket

Bed samples—These were collected in the pocket and the canal.

Boil samples—These were obtained by bottle samplers in the boils of the canal regulator bays and undersluice bays.

Special samples—While analysing the data obtained during 1940-41 it was noticed that the efficiency of the silt excluders as worked out by Mr. Haigh's Formula dropped suddenly when the discharge in the river went above 100,000 cusecs. This drop in the efficiency was thought to be due to the intensive mixing of the suspended silt due to the extra turbulence in the river water during discharges higher than 100,000 cusecs. It was decided to verify this assumption. Hence it was arranged to take bottle samples 5 times during the course of the next summer for the following groups of upstream river discharges, 80–90,000 cusecs, 90—100,000 cusecs, 100,000—110,000 cusecs, 110—130,000 cusecs, and above 130 000 cusecs. Whil these observations were being taken it was also noted whether the river was rising or falling. These bottle samples (10 bottles per sample) were collected at R. D. 300 of the excluder pocket on the centre vertical before each tunnel at every ·1D. These are given in Figs. 205-206. These curves do not justify the above assumption. Further observations will be taken during 1942 in this connection.

In analysing the data for 1940-41, the following index—

$$\text{Discharge ratio} = \frac{\text{Canal Discharge}}{\text{Tunnel Discharge}} \times 100 = \text{D. R.}$$

was found to be directly correlated with the efficiency of the silt ejectors. During 1939 and 1940 this had been kept between 33 to 80 per cent for the left canal and 10 to 12 per cent for the right channel. During 1941-42 it was not possible to have any bigger range of D. R.

Special Observations—Velocity observations (vertical) were carried out before each tunnel in the left excluder pocket at R. D. 300. This was done for different regulations of the undersluice gates passing different discharges and maintaining different pond levels. All these data have been given in Table XXVI.

Right Pocket

Bed samples—These were collected in the pocket and the canal.

of the Lower Chenab Canal system due to the construction of silt excluder tunnels at Khanki Headworks. The immediate effect of this silt excluder had been to scour the head reach of the Main Line. Part of this scoured silt is to be found in the tail of the Main Line above Sagar. At Sagar the Upper Gugera Branch takes off. This branch also has scoured in the head reach and deposited part of the silt in the tail reach above Buchiana. At Buchiana the channel bifurcates into [illegible] and Burala Branch. Both the [illegible] silt movement in the head reach [illegible] the tail reach than at the head. [illegible] as the Main Line Lower. This channel is all throughout in scour in its 12 miles length. At Nanuana the Main Line Lower trifurcates into Jhang Branch, Rakh Branch and the Mian Ali Branch. Jhang Branch shows signs of moderate silt movement all throughout its length except in the head reach. The Rakh Branch shows almost no sign of silt movement except in the head reach where it is in scour. Mian Ali Branch also appears to have the same amount of silt movement as the Jhang Branch except that it has no scour in the head reach.

The above account of bed silt movements for the different channels is supported by observation of the slope of the water surface in the different reaches of these channels. In Tables XXXIV—XLI are given the values of actual slopes as observed during 1940-41 and 1941-42 kharif; along side are also given the values of the slope calculated from the following equation—

$$S \times 10^3 = 2{\cdot}09 \frac{m^{\cdot 86}}{Q^{\cdot 21}}.$$

Main Line Upper—There is very little change in the observed slope in the reach. In the head reach the agreement between the observed and the calculated slope is much better, while in the tail reach the observed slope is much steeper which indicates silting.

Upper Gugera Branch—The agreement between the calculated and the observed slopes is very close in the head reach and becomes worse in the tail reach. There is some slight change in the observed slope in the tail reaches.

Lower Gugera Branch—The observed slopes are much steeper than the calculated slopes indicating heavy bed silt movements. Between 1940 and 1941 observations there is practically no change.

Burala Branch—Here also the observed slope is very much steeper than the calculated one, corroborating the observations noted before that there is heavy bed silt movement in the reach. Comparison of the slopes observed during 1940 and 1941 indicates that the silting that was noticed in the reach between R. D. 59,500—105,000 during 1940 has become worse during 1941. There is also indication of heavy

silt movement in the tail reach downstream of the fall at R. D. 180,000.

Jhang Branch Upper—The observed slopes are steeper than the calculated slopes as in 1940. There is very little change in the observed slopes between 1940 and 1941.

Jhang Branch Lower—The observed slopes are steeper than the calculated ones as in 1940. Excepting in one or two reaches the slopes are the same as during 1940.

Rakh Branch—The observed and the calculated slopes are very close to each other and there is no change in the observed slopes between 1940-41.

Mian Ali Branch—Here the observed slopes are slightly in excess of the observed slopes. There appears to have taken place some scouring in the tail reach during 1941.

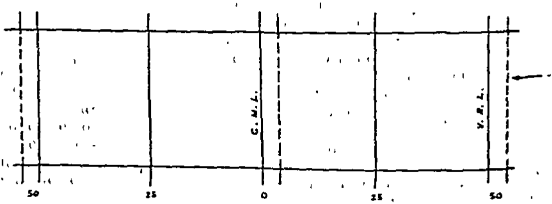
To follow the changes more closely, monthly observations were carried out in the Rakh Branch and the Lower Gugera Branch. Excepting in a few reaches there is no indication to show that there is any periodic change in the slopes of the water surface in these two channels. These have been given in Tables XLII—XLIII.

Comparison of Canal Discharges as measured by the current meter and the Lacey type velocity rod—The method of taking discharges in a canal or a river by current meters is to divide the whole cross section into a number of segments by drawing verticals. The current meter is placed at ·6 depth of the central vertical of each of these segments. The flow through each segment is found by multiplying the velocity so obtained by the area of the segment. It has been accepted that the velocity found at ·6 depth gives the average value of the velocities over the whole depth in that vertical. It is contended by many that when the depth is small, say 1·5′ or 2′, this method of taking velocities is defective and it is preferable to find velocities by velocity rods. The experiments were carried out at different regime sites in the Punjab. The following channels were selected in order to compare observations for different ranges of discharges :—

1.	Tarkhani Distributary .	R. D. 19,500, Discharge 190 cusecs.
2.	Hafizabad Distributary ..	R. D. 3,116, Discharge 20·0 cusecs
3.	Lakhuana Distributary ..	R. D. 12,000, Discharge 99·7 cusecs.
4.	Rakh Branch ..	R. D. 231,500, Discharge 414·2 cusecs.
5.	Shahkot Distributary ..	R. D. 12,000, Discharge 162·3 cusecs.
6.	Mian Ali Branch ..	R. D. 95,000, Discharge 387·7 cusecs.
7.	Mian Ali Branch ..	R. D. 9,500, Discharge 551·6 cusecs.
8.	Udhoki Distributary ..	R. D. 3,500, Discharge 10·1 cusecs.
9.	Lower Gugera Branch ..	R. D. 11,000, Discharge 1,829·3 cusecs.
10.	Main Line Lower L.C.C. ..	R.D. 149,400, Discharge 1,163·1 cusecs

Since velocity rod observations take a much longer time than required by the current meter, the experiments were arranged in such a way that the two sets of observations with current meter and velocity rods went on simultaneously. In this way while the velocity rods gave one value of the discharges the current meter gave two or more values. The ultimate object of all such velocity observations being the calculation of discharges, it was thought advisable to compare discharges as observed by current meter and velocity rods simultaneously. If the velocity of flow at a certain point of the channel could be observed by the velocity rod and the current meter at the same time then the method of comparing velocities would have been ideal. As it is physically impossible to do this, the following method was adopted for observing velocities with the current meter and the velocity rod simultaneously and then calculating the discharges over the period of observation.

The stretch of the canal selected for carrying out these observations was properly berm trimmed for a distance of 500 feet upstream and downstream of the central section. Five wire ropes were stretched across the channel as shown in the sketch below :—



The centre line was meant for current meter observations hence it was provided with red pendants and a subsidiary wire at a distance of 10 feet for holding the boat. The other wires were at 25 feet and 50 feet upstream and downstream of the central line. The 50 feet upstream wire was provided with a subsidiary wire for holding the boat from which the velocity rods were released. Pendants (red) were attached to all the five wires at the same points as the central

line. On both sides of each red pendant white pendants were fixed at a distance of 1 foot to 1·5 feet. It was found by practice that while releasing the velocity rod, the rod had to be kept at least 3 to 4 feet upstream of the 50 feet upstream wire, otherwise while crossing the first upstream wire the motion of the velocity rod was not steady. Timing was started when the velocity rod crossed the first upstream wire. The times of crossing by the rod each of the wires at 25 feet upstream, centre, 25 feet downstream and 50 feet downstream were noted by an overseer. If the rod in passing the wires happened to stray beyond the two white marks for the particular rod pendant at which it was dropped, the run was rejected. The rod was recovered from the water at about 10 feet downstream of the 50 feet downstream wire. Three good runs were obtained for each pendant point. The gauge at site was noted after observations for each pendant point were completed. Observations with the current meter were carried out on the central line and were started from the left of the canal. When the current meter velocity observations across the section were finished from the left by one overseer, the current meter velocity observations were started from the right by the overseer who had previously been carrying out velocity rod observations. Thus after every set of current meter velocity observations the overseers were interchanged but the observations proceeded without any loss of time. Current meter observations were carried out until the velocity rod observations were completed.

From the above it will be seen that the comparison was really between the discharge that was flowing in the canal for a certain length of time as measured by the current meter and the velocity rod. These observations were carried out for two different lengths of the rod, ·8 and ·9 of the depths at different pendants. These experiments were carried out with Lacey Type velocity rod lent to the Institute.

Analysis of Data—The results of these observations have been calculated and tabulated (Tables XLIV—XLVIII). As each run was for distances 100 feet and 50 feet, two values of the velocities have been obtained by the velocity rod for each pendant point. Corresponding to these two velocities two discharges have been calculated and called $Q_{100'}$, and $Q_{50'}$. Similarly while the velocity rod observations were being carried out the current meter observations were taken twice over the cross section. There are, therefore, two values for discharge by the current meter also.

These data will now be analysed more closely.

1. *Hafizabad Distributary*—The depth in the cross section varies from 1·7′ to 2·05′ and the average discharge is 21 cusecs. Current meter observations show that the discharge was steady within 2 per cent during the course of each set. Discharges by velocity

for runs 50 feet and 100 feet were within 2 per cent of each other. Discharges by velocity rod for both ·8 and ·9 depths were higher than the dicharges by the current meter at ·6 depth. The difference varies between 7 and 11 per cent.

2. *Lakhuana Distributary*—The depth in the cross section varies from 1·7 feet to 2·5 feet and the average discharge is about 110 cusecs. Current meter observations show that the discharge was steady within 2 per cent during the course of each set. Discharges by velocity rod for runs 50 feet and 100 feet were within 3 per cent of each other. Discharges by velocity rod ·8 depth were always higher than those by the current meter up to about 10 per cent and for ·9 depth rod they were sometimes more and sometimes less than those by the current meter. On an average ·9 depth rod gives discharges very close to those by the current meters.

3. *Tarkhani Distributary*—The depth varied from 2·0 feet to 3·0 feet and the discharge was about 190 cusecs. For ·8 depth velocity rod the discharges are less than those by the current meter on an average.

4. *Rakh Branch*—The depth varied from 2·9 feet to 4·2 feet and the discharge was about 400 cusecs. Discharges by the velocity rod of ·9 depth are always less than those by a current meter up to about 5 per cent and for ·8 depth velocity rod they are sometimes more and sometimes less.

5. *Lower Gugera Branch*—The depth varied from 5·0 feet to 7·0 feet and the discharge was about 1,700 cusecs. Velocity rod of ·8 depth always gave higher discharges than the current meter up to a limit of 10 per cent. With rods of ·9 depth the discharges are sometimes more and sometimes less than those given by the current meter.

Conclusion—Discharges by the velocity rod of ·8 depth always gave higher values than those by the current meter at ·6 depth. The maximum difference is about 10 per cent. It is not possible from these sets of data obtained to arrive at any regression between the current meter discharge and the velocity rod discharge (·8 depth).

(ii). Discharges by the velocity rod of ·9 depth when averaged over a number of observations give the same value as that given by the current meter at ·6 depth. But observations with ·9 depth velocity rod are not reliable as very often the bottom end of the rod touches the bed of the canal which is generally uneven.

Report on the working of the B. S. P. pipes and the analysis of daily B. S. P. observations during the year 1940.

The B. S. P. pipes were put in by Mr. Crump while on special duty in connection with the waterlogging investigation during 1938—40. Regular observations of the water levels recorded by these pipes were

started in early 1940. The following number of pipes were observed daily for different Circles :—

1.	Lower Chenab Canal	22
2.	Haveli Canal	1
3.	Upper Chenab Canal	8
4.	Lower Bari Doab Canal	1
5.	Lower Chenab Canal	2

According to Mr. Crump these pipes should record the true spring level at the site and this should be unaffected by the presence of the canal or rainfall. Their only value lies in this supposed characteristic. If the pipes do not satisfy this criterion their readings would be misleading. To determine whether these pipes satisfy Crump's requirements, their daily readings have been examined and the results of the examination are given below. From this it will be seen that a number of these pipes appear to be choked and others respond to variation in canal supplies or to rainfall. In order to be sure that the pipes that do not respond immediately to canal closures or rainfall but show some slight rise or fall were in order, arrangements were made to pump them out and test their responsiveness. The responsiveness was tested in the following way. Each pipe was filled with water to the top and then the water was allowed to fall. In the case of good pipes the water went down very quickly, even in less than one or two minutes. For pipes partially choked it generally took 15 minutes to one hour and for others almost completely choked it took between three to four hours. After this test, the pipes were pumped out and the test again applied. If the test after pumping was found satisfactory the pipe has been classed as satisfactory from the point of view of recording water level.

Lower Chenab Canal Circle

1. *Khanki*—Here the pipe reading varies with the river and not with the canal. The pipe is not recording the unaffected spring level.

2. *Sagar*—Here the pipe varies with the canal and each time the canal is closed the B. S. P. level goes down. Rainfall does not seem to have any influence on it. The pipe does not record true spring level.

3. *Chuharkana*—Here the pipe readings follow the canal levels closely. Rainfall does not seem to have much influence. The pipe does not record true spring level.

4. *Pacca Dalla*—Here the B. S. P. level is 15 feet below the canal full supply level and does not respond to its fluctuations. This pipe was, therefore, pumped out and the responsiveness tested after pumping. The pipe was found satisfactory and is recording unaffected spring level.

5. *Hinduana*—There is a difference of 18 feet between the B. S. P. level and the canal full supply level. Canal closures are reflected by drops in the B. S. P. level. The pipe does not record true spring level.

6. *Kot Ahmad Yar Khan*—There is a difference of more than 20 feet between the B. S. P. level and the canal full supply level. Canal closures have no effect on the B. S. P. readings. Seasonal fluctuations are reflected very slightly by the B. S. P. levels. The pipe appeared to be choked. It was tested for responsiveness and it was found that the water level went down quickly. On lowering the sounder it struck against something in the pipe one inch below the water level hence no fluctuations in the water level were recorded. The pipe is otherwise reliable.

7. *Kot Khuda Yar*—There is a difference of 6 to 8 feet between the B. S. P. level and the canal full supply level. Complete canal closures affected the B. S. P. readings. The pipe appeared to be partially choked. After pumping it was completely responsive. The pipe does not record true spring level.

8. *Lyallpur*—There is a difference of 45 feet between the B. S. P. level and the canal full supply level. Neither canal closures nor rainfall have any effect on the B. S. P. readings. The pipe appeared to be partially choked. As the water table was too deep no pumping could be done. The responsive test showed that the pipe is partially choked. It should be pumped out before it can be used.

9. *Tawan*—There is a difference of about 42 feet between the B. S. P. level and the canal full supply level. Canal closures or rainfall have no effect on the B. S. P. readings. As the water level was very deep no pumping could be done. Responsive test showed that the pipe was reliable, and recording true spring level.

10. *Muradwala*—There is a difference of 24 feet between the B. S. P. and full supply level of the canal. Neither canal closures nor rainfall have any effect on the B. S. P. readings. Responsive test before pumping showed that the pipe was partially choked. After pumping it became reliable. Readings taken by the local overseer appear to have been done carelessly.

11. *Lakhlana*—There is a difference of 33 feet between the B. S. P. and full supply level of the canal. Neither canal closures nor rainfall have any effect on the B. S. P. readings. Responsive test before pumping showed partial choking. As the water table was very deep no pumping could be done.

12. *Janiwala*—There is a difference of about 44 feet between the B. S. P. and the full supply level of the canal. Neither canal closures nor rainfall have any effect on the B. S. P. readings. Responsive test before pumping showed partial choking. As the water level was deep no pumping could be done.

13. *Varyam*—There is a difference of about 23 feet between the B. S. P. and full supply level of the distributary. Neither canal closures nor rainfall have any effect on the B. S. P. readings. Responsive test before pumping showed partial choking. After pumping the pipe appeared to be reliable and recording true spring level.

14. *Buchiana*—There is a difference of about 17 feet between the B. S. P. and full supply level of the canal. There seems to be some slight response to canal closures. Responsive test before pumping showed choking. After pumping the pipe was responsive. The pipe is not reading true spring level.

15. *Tarkhani*—There is a difference of 36 feet between the B. S. P. and the canal full supply level. The B. S. P. levels follow canal closures. The pipe is not recording true spring level.

16. *Muriduala*—There is a difference of 36 feet between the B. S. P. level and distributary full supply level. The B. S. P. levels do not follow the canal closures. Responsive test before pumping showed partial choking. As the water level was deep no pumping could be done.

17. *Magneja*—There is a difference of 20 feet between the B. S. P. level and distributary full supply level. The B. S. P. levels do not follow the canal closures. Responsive test before and after pumping showed that the pipe was unreliable.

18. *Bachrianwala*—There is a difference of 37 feet between the B. S. P. and full supply level. The B. S. P. readings do not follow the canal closures. Responsive test before pumping showed that the pipe was partially choked. As the water level was deep no pumping could be done.

19. *Khai*—There is a difference of 27 feet between the B. S. P. level and distributary full supply level; but a difference of only one foot between the B. S. P. level and the lowest river level. The B. S. P. levels follow the river very closely. The pipe is not recording true spring level.

20. *Sandianwala*—There is a difference of 29 feet between the B. S. P. and the distributary level. In 1941 there appears to be some response to the river. The pipe is not recording true spring level.

21. *Sultanpur*—There is a difference of about 20 feet between the B. S. P. level and the full supply level. The B. S. P. readings do not follow the distributary level. Responsive test before and after pumping showed that the pipe was choked. The pipe is unreliable.

22. *Nabipur*—There is a difference of 23 feet between the B. S. P. and the distributary level. In the early part of the year the B. S. P. did not respond to canal closure but a closure on 24th October 1940 for 12 days produced a marked drop in the B. S. P. level. Responsive test before and after pumping showed that the pipe was choked. The pipe is unreliable.

HAVELI CANALS CIRCLE

1. *Trimmu*—There is a difference of 3 to 4 feet between the canal full supply and the B. S. P. level. The canal closures do not have any effect on the B. S. P. levels. Rainfall appears to have an immediate effect but the river level seems to be the predominating factor in determining the B. S. P. level. The pipe is not recording true spring level.

UPPER CHENAB CANAL

1. *Marala*—There is a difference of about 3 feet between the highest B. S. P. level and the canal level and a difference of 5 feet between the B. S. P. level and the river level. The B. S. P. levels do not follow the canal or the river. Responsive test before pumping showed that the pipe was partially choked. After pumping the pipe is reliable.

2. *Khambrianwala*—There is a difference of 6 feet between the maximum B. S. P. level and the canal level. The B. S. P. level follows the canal closures. The pipe is not recording true spring levels.

3. *Sangowali*—There is difference of 11 feet between the canal full supply and the B. S. P. maximum level. The B. S. P. levels do not respond to canal closures nor to rainfall. Responsive test before and after pumping showed that the pipe is recording true spring level.

4. *Gujranuala*—There is a difference of 13 feet between the highest B. S. P. level and the canal full supply level. The B. S. P. pipe does not seem to respond to canal closures nor to rainfall. Responsive test before and after pumping showed that the pipe is reliable.

5. *Badoratta*—There is a difference of 11 feet between the highest B. S. P. level and the canal level. The drain is about 4 feet higher than the highest B. S. P. level. The B. S. P. level is unaffected by canal closures but is affected by rainfall. Responsive test before pumping showed that the pipe was partially choked. The pipe is unreliable.

6. *Chichoki Malian*—The difference between the B. S. P. and the canal full supply level is 4·5 feet. The B. S. P. level responds to canal supply. It is not very sensitive to local rainfall. The pipe is not recording true spring level.

7. *Mangtanwala*—There is a difference of 15 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does not respond to fluctuations of the canal supply. Responsive test before pumping showed that the pipe was partially choked. After pumping it is reliable.

8. *Baraghar*—There is a difference of 17 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does

not respond to canal closure nor the local rainfall. Responsive test before pumping showed that the pipe is partially choked. After pumping it is reliable.

Lower Bari Doab Canal Circle

1. *Balloki*—There is a difference of 4 feet between the B. S. P. level and the canal full supply level. They follow each other very closely. The response of the B. S. P. levels to local rainfall is not very marked. The pipe does not record undisturbed spring level.

2. *Hala*—There is a difference of 11 feet between the B. S. P. level and the canal full supply level. The B. S. P. level responds quickly to canal supply level. The pipe does not record true spring level.

3. *Montgomery*—There is a difference of 33 feet between the B. S. P. and the canal full supply level. The B. S. P. levels do not respond to canal fluctuations or local rainfall. There is a sudden change in the B. S. P. level between 16th October to 16th November 1940. This must be due to some change in the measuring arrangement. Responsive test before and after pumping showed that the pipe is choked. The pipe is unreliable.

4. *Renala*—There is a difference of 18 feet between the B. S. P. and the canal full supply level. The B. S. P. level responds very slightly to canal fluctuations, but not to local rainfall. The pipe does not record true spring level.

Lower Jhelum Canal Circle

1. *Sargodha*—Canal levels were not plotted. The pipe was reported by the Executive Engineer as being choked. It has been recently taken up, cleaned and put back. Responsive test showed that the pipe is working now satisfactorily.

2. *Faquirian*—No B. S. P. observations were taken during 1940. Responsive test before pumping showed that the pipe was partially choked. After pumping it was responsive.

Conclusion—In the attached Table XLIX these pipes have been classified as—

(i) *Satisfactory*, i.e. they indicate the true B. S. P. level. In this category there are two classes of pipes,—

(a) One that has been satisfactory from the very beginning.

(b) the others that were choked previously and may prove satisfactory now after pumping if observations are carried on for some time.

This class includes :—

(a) Pipes Nos. 9, 26, 27 ;

(b) Pipes Nos. 4, 6 (if the obstacle is removed), 8, 10, 11, 12, 13. 16, 17 18, 21, 22, 24, 30, 31, 34, 36 and 37.

HAVELI CANALS CIRCLE

1. *Trimmu*—There is a difference of 3 to 4 feet between the canal full supply and the B. S. P. level. The canal closures do not have any effect on the B. S. P. levels. Rainfall appears to have an immediate effect but the river level seems to be the predominating factor in determining the B. S. P. level. The pipe is not recording true spring level.

UPPER CHENAB CANAL

1. *Marala*—There is a difference of about 3 feet between the highest B. S. P. level and the canal level and a difference of 5 feet between the B. S. P. level and the river level. The B. S. P. levels do not follow the canal or the river. Responsive test before pumping showed that the pipe was partially choked. After pumping the pipe is reliable.

2. *Khambrianwala*—There is a difference of 6 feet between the maximum B. S. P. level and the canal level. The B. S. P. level follows the canal closures. The pipe is not recording true spring levels.

3. *Sangowali*—There is difference of 11 feet between the canal full supply and the B. S. P. maximum level. The B. S. P. levels do not respond to canal closures nor to rainfall. Responsive test before and after pumping showed that the pipe is recording true spring level.

4. *Gujranwala*—There is a difference of 13 feet between the highest B. S. P. level and the canal full supply level. The B. S. P. pip does not seem to respond to canal closures nor to rainfall. Responsive test before and after pumping showed that the pipe is reliable.

5. *Badoratta*—There is a difference of 11 feet between the highest B. S. P. level and the canal level. The drain is about 4 feet higher than the highest B. S. P. level. The B. S. P. level is unaffected by canal closures but is affected by rainfall. Responsive test before pumping showed that the pipe was partially choked. The pipe is unreliable.

6. *Chichoki Malian*—The difference between the B. S. P. and the canal full supply level is 4·5 feet. The B. S. P. level responds to canal supply. It is not very sensitive to local rainfall. The pipe is not recording true spring level.

7. *Mangtanwala*—There is a difference of 15 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does not respond to fluctuations of the canal supply. Responsive test before pumping showed that the pipe was partially choked. After pumping it is reliable.

8. *Baraghar*—There is a difference of 17 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does

not respond to canal closure nor the local rainfall. Responsive test before pumping showed that the pipe is partially choked. After pumping it is reliable.

Lower Bari Doab Canal Circle

1. *Balloki*—There is a difference of 4 feet between the B. S. P. level and the canal full supply level. They follow each other very closely. The response of the B. S. P. levels to local rainfall is not very marked. The pipe does not record undisturbed spring level.

2. *Hala*—There is a difference of 11 feet between the B. S. P. level and the canal full supply level. The B. S. P. level responds quickly to canal supply level. The pipe does not record true spring level.

3. *Montgomery*—There is a difference of 33 feet between the B. S. P. and the canal full supply level. The B. S. P. levels do not respond to canal fluctuations or local rainfall. There is a sudden change in the B. S. P. level between 16th October to 16th November 1940. This must be due to some change in the measuring arrangement. Responsive test before and after pumping showed that the pipe is choked. The pipe is unreliable.

4. *Renala*—There is a difference of 18 feet between the B. S. P. and the canal full supply level. The B. S. P. level responds very slightly to canal fluctuations, but not to local rainfall The pipe does not record true spring level.

Lower Jhelum Canal Circle

1. *Sargodha*—Canal levels were not plotted. The pipe was reported by the Executive Engineer as being choked. It has been recently taken up, cleaned and put back. Responsive test showed that the pipe is working now satisfactorily.

2. *Faquirian*—No B. S. P. observations were taken during 1940. Responsive test before pumping showed that the pipe was partially choked. After pumping it was responsive.

Conclusion—In the attached Table XLIX these pipes have been classified as—

(i) *Satisfactory*, i.e. they indicate the true B. S. P. level. In this category there are two classes of pipes,—

(a) One that has been satisfactory from the very beginning.

(b) the others that were choked previously and may prove satisfactory now after pumping if observations are carried on for some time.

This class includes :—

(a) Pipes Nos. 9, 26, 27 ;

(b) Pipes Nos. 4, 6 (if the obstacle is removed), 8, 10, 11, 12, 13, 16, 17 18, 21, 22, 24, 30, 31, 34, 36 and 37.

HAVELI CANALS CIRCLE

1. *Trimmu*—There is a difference of 3 to 4 feet between th canal full supply and the B. S. P. level. The canal closures do nc have any effect on the B. S. P. levels. Rainfall appears to have a immediate effect but the river level seems to be the predominatin factor in determining the B. S. P. level. The pipe is not recording tru spring level.

UPPER CHENAB CANAL

1. *Marala*—There is a difference of about 3 feet between th highest B. S. P. level and the canal level and a difference of 5 fee between the B. S. P. level and the river level. The B. S. P. level do not follow the canal or the river. Responsive test before pumpin showed that the pipe was partially choked. After pumping the pipe i reliable.

2. *Khambrianwala*—There is a difference of 6 feet betwee the maximum B. S. P. level and the canal level. The B. S. P. leve follows the canal closures. The pipe is not recording true sprin levels.

3. *Sangowali*—There is difference of 11 feet between the cana full supply and the B. S. P. maximum level. The B. S. P. levels dc not respond to canal closures nor to rainfall. Responsive test befor and after pumping showed that the pipe is recording true sprin level.

4. *Gujranwala*—There is a difference of 13 feet between the highest B. S. P. level and the canal full supply level. The B. S. P. pip does not seem to respond to canal closures nor to rainfall. Responsive test befor and after pumping showed that the pipe is reliable.

5. *Badoratta*—There is a difference of 11 feet between the highest B. S. P. level and the canal level. The drain is about 4 feet higher than the highest B. S. P. level. The B. S. P. level is unaffected by canal closures but is affected by rainfall. Responsive test before pumping showed that the pipe was partially choked. The pipe is unreliable.

6. *Chichoki Malian*—The difference between the B. S. P. and the canal full supply level is 4·5 feet. The B. S. P. level responds to canal supply. It is not very sensitive to local rainfall. The pipe is not recording true spring level.

7. *Mangtanwala*—There is a difference of 15 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does not respond to fluctuations of the canal supply. Responsive test before pumping showed that the pipe was partially choked. After pumping it is reliable.

8. *Baraghar*—There is a difference of 17 feet between the B. S. P. level and the canal full supply level. The B. S. P. level does

Of these pipes the following:—8, 11, 12, 16, 17, 18, 21, 22, 34 have to be taken out and cleaned properly before they would be able to respond to the changes in the water level. They are choked and could not be improved by pumping.

(*ii*) The rest can be declared as useless for the purposes for which they were put in. From this it will be seen that only the pipes at Tawan, Sangowali and Gujranwala had been behaving as true B. S. P. pipes. Observation of the water level records of these pipes for 1940 will be examined in detail now.

Pipe at Tawan Rest House—The B. S. P. level attains its maximum value in October and its minimum in February. The difference in level is about 2 feet and takes place gradually from February to October—so that the rise is spread over a period of 8 months while the fall takes place in 4 months.

Pipe at Sangowali Rest House—The B. S. P. level attains its maximum value in September and the minimum in April. The difference in level is about 5 feet and takes place gradually up to the end of July and then suddenly goes up by 3 feet. Here the rise is spread over 5 months while the fall is more gradual over 7 months.

Pipe at Gujranwala Rest House—In this case there appear to be two maxima—one spread over February and March and another over November and December, minimum being attained in the month of July. The difference between the maximum and minimum level is about 2 feet.

The case of Gujranwala B. S. P. pipe seems to be peculiar for the place. It is in a waterlogged area where pumping is going on continuously. The two humps may be due to pumping.

The two pipes, one at Sangowali Rest House and another at Tawan Rest House, are situated almost at the two ends of the Rechna Doab, the first one being very close to the foot hills and being in a region where the water-table is very high. In this pipe the maximum B. S. P. level is recorded in September. The other pipe is situated at almost the tail of the Doab where the water-table is very deep. In this pipe the maximum B. S. P. is recorded in October. This difference in time between the two maxima may be due to the fact that the pressure wave started at the upper end in the foothill of the Himalayas due to monsoon downpour is recorded by the first pipe much earlier than the second pipe. The progress of the pressure wave could be followed more clearly if all the pipes were recording truly.

For this purpose observation of the following pipes may be continued :—

Pacca Dalla, Kot Ahmad Yar Khan, Lyallpur, Tawan, Muradwala, Janiwala, Varyam, Muridwala, Sultanpur, Nabipur, Marala, Sangowali, Gujranwala, Mangtanwala, Baraghar, Bachrjanwala, and Magneja.

The pipes at the following places have to be taken out, cleaned and put back before observations can be started :—

Lyallpur, Janiwala, Muridwala, Sultanpur, Nabipur, Bachrianwala, and Magneja.

Experiments on Seepage Discharge from canals into drains and vice versa—The laws of seepage flow from the canal to a drain or from a drain to a canal have not been so far worked out mathematically. Though the laws will follow Laplace's Equation as the flow is in a saturated porous medium, yet as the free surface connecting the canal to the drain is of an unknown configuration it is not possible to apply the method of conformal transformation to this particular problem. A very specialized case of seepage from a sloping spring level to a drain has been worked out by Hopf (Zeitschrift für Angewandte Mathematik und Mechanik, 1921). Daehler had also tried to solve this problem by combining experimental results with theoretical analysis ("Über die Versicherung aus Kanälen", "Die Wasserwirtschaft" 1933). As none of these methods lead to any satisfactory solution of the problem it was desired to set up a series of experiments in which—

(1) heads comparable to those found in nature could be tried ;

(2) the distance between the drain to the canal would be such that for the highest head tried the drawdown produced by the drain will not affect the seepage from the canal ;

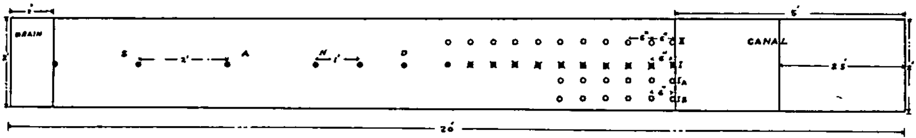
(3) the flow from the canal to the drain could be easily reversed to that from the drain to the canal ;

(4) the effect of the impermeable bottom will not affect the stream lines.

A reinforced concrete tank whose internal dimensions are $20' \times 2' \times 10'$ was built and filled up with sand that was sieved between two definite grades. Samples were taken from every 6 inches of the depth as the tank was filled and analysed for average diameter size and transmission constant. The average diameters varied from ·28 to ·32 mm and the transmission constant of a mixture of all the samples was determined and found to be ·00035 feet/sc. at 20°C. A number of pressure pipes each ½ inch diameter were installed at different points of the medium. This is shown in Fig. 207. It will be seen that a line of straight pipe 8 feet long were arranged as a central row No. 1 in the tank. The eleven pipes nearest to the canal were perforated the whole length so that they worked as strainers. They were spaced at 6 inches intervals. After this 3 pipes were spaced at intervals of 1 foot and had strainers to a height of 2 feet from the bottom. The remaining three pipes were spaced at 2 feet intervals and had strainers only at the bottom. The last pipe was almost touching the drain. Besides these there were three rows of pipes with strainers at the end—one row consisting of 6 pipes completely below the bed of the canal Row No. 1-A ; another row also of 6 pipes at the corner between the bed

FIG. 207

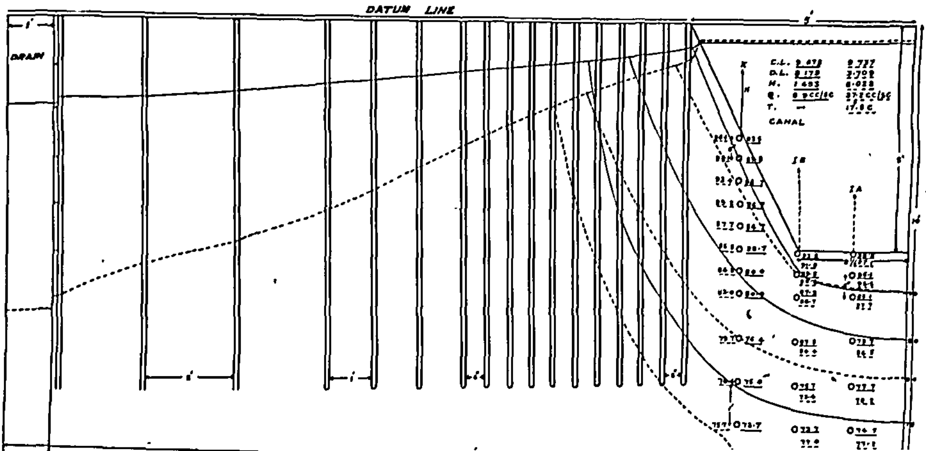
PLAN OF THE SEEPAGE TANK



Row I pipes ▣ perforated upto 7' from the bottom (5 holes at every 3" distance) ○.
" ▣ " " 5' " "
" ● " " 2' " "
Rows II, IA, IB, strainers at the ends only, these go under the bed of the canal.

FIG. 208
SEEPAGE TANK

DATE 2/42



and the berm (Row No. 1B) and the third row of 11 pipes half way up the berm, Row No. II. These pipes were bent and brought up on the surface. A pair of straight edges run along the top of the tank from which water-levels are measured by means of a special arrangement. The R. L. of the straight edge was 10.

The row No. 1 served to indicate the free water surface connecting the canal to the drain. Row No. 1A, 1B and II helped to draw the equi-pressure lines below the bed of the canal. Two cases of drain levels with the same canal levels have been plotted in the Figs. 208—209. The canal level was the highest that was available in the model. Two cases one with 1·493 feet and another 6·028 feet heads have been shown in Fig. 209. The free surface with the highest head 6·028 feet shows kinks which are due to the high drawdown induced by the drain.

These experiments have been arranged in the following way :—

(1) Canal bed level fixed. For different canal supply levels the head from the canal to the drain has been varied from about 1 foot to 7·5 feet. For each head the discharge, temperature and pressures were observed. At least seven to eight heads were observed for each canal level. It has been possible to do two canal bed levels R. L. 5·0 and R. L. 6·0 up *till* now.

(2) Canal bed level fixed. For different drain supply levels the head from the drain to the canal has been varied from about 1 foot to 4·5 feet. With the canal bed level fixed at R. L. 5·0, it is not possible to have any higher head.

It is well known that the viscosity of water diminishes with the rise of temperature. Hence at higher temperatures there will be more discharge than for lower temperatures, other factors remaining steady. Hence discharges in all these cases have been reduced to 20°C and have been expressed in cc/sc. These have been tabulated in Tables L—LI and plotted in Fig. 209. From this figure it will be seen that the discharges from the canal into the drain follow the following *relationship* :—

$$Q_{20} = C.h$$

where h is the head in feet and Q is cc/sc. Similarly the discharge *from the drain into the canal follows* the same law.

Experiments under different conditions are being continued.

FIG. 209

MODEL EXPERIMENT ON SEEPAGE FOR CANAL TO DRAIN & VICE VERSA

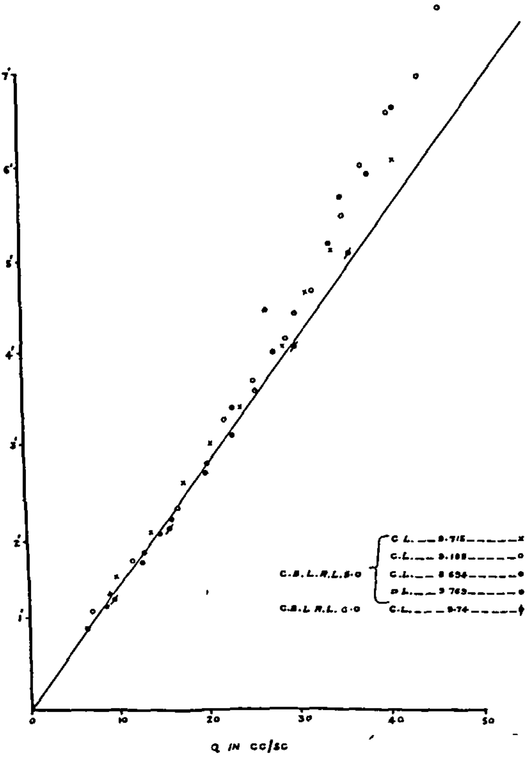


FIG. 2.09

MODEL EXPERIMENT ON SEEPAGE FOR CANAL TO DRAIN & VICE VERSA

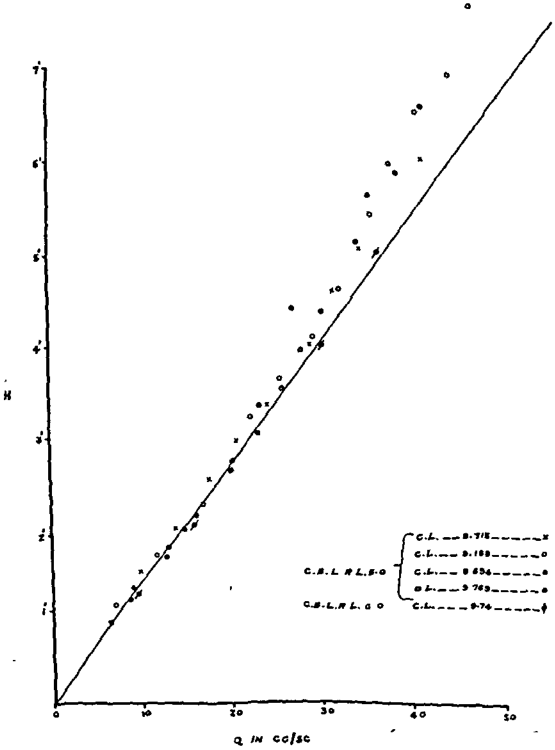


FIG. 209

MODEL EXPERIMENT ON SEEPAGE FOR CANAL TO DRAIN & VICE VERSA

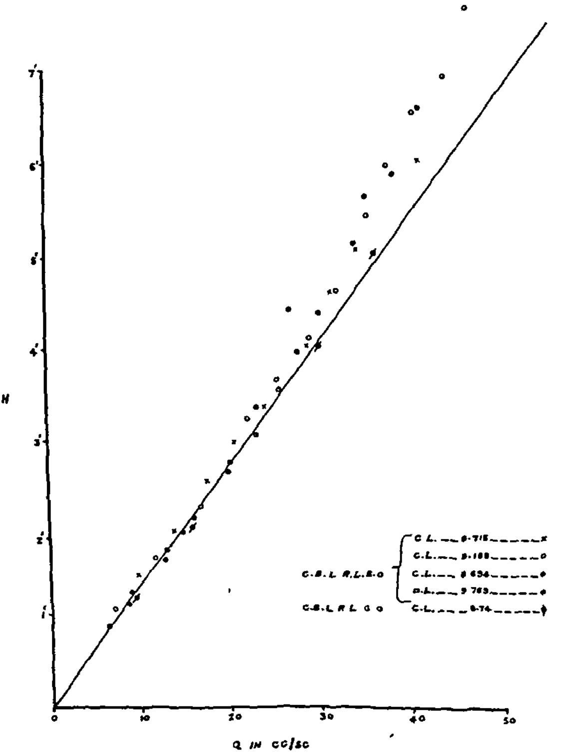


TABLE XXII

STATEMENT SHOWING THE AVERAGE VALUES OF THE HYDRAULIC DATA FOR CURRENT SITES

Site	Channel	R D	Period	Q	A	S × 10³	R	D	P	N	M × 10³
Nanuana	Jhang Branch ..	7,260	November 1934 to December 1941	2,827·35	987·30	·218	6·81	7·58	144·97	2·87	42·65
	Rakh Branch	(a) 7,260	November 1934 to December 1940	1,056·31	428·03	·208	4·72	5·25	90·61	2·54	40·90
		(b) 7,260	January 1941 to December 1941	1,148·77	446·49	·207	4·70	5·36	93·29	2·57	40·13
	Udhoki Distributary	(a) 2,372	May 1937 to December 1940	40·66	26·83	·305	1·49	1·80	17·97	1·52	25·81
		(b) 2,372	January 1941 to December 1941	37·93	26·93	·298	1·33	1·61	19·48	1·41	22·97
Sagar	Main Line Lower ..	(a) 149,000	October 1933 to December 1940	4,473·20	1,535·67	·191	7·55	8·12	203·57	2·92	42·70
		(a) 149,000	January 1941 to December 1941	4,617·57	1,579·03	·203	7·69	8·33	205·38	2·91	42·71 42·17
	Upper Gugera Branch	2,500	June 1934 to December 1941	4,952·53	1,619·34	·143	8·80	10·54	182·79	3·06	42·10
	Hafizabad Distributary	3,050	January 1941 to December 1941.	23·90	16·96	·239	1·43	1·97	11·90	1·41	24·91
Ughana	Rakh I	213,880	October 1939 to December 1941	581·57	257·21	·140	3·93	4·51	64·95	2·26	24·98
	Rakh II	235,620	October 1939 to December 1941	416·98	197·22	·167	3·38	3·74	58·79	2·11	24·30
	Lakhuana Distributary	10,000	October 1939 to December 1941	105·11	61·89	·230	1·98	2·25	31·21	1·70	24·02
Pacca Dalla	Mian Ali Branch ..	95,000	May 1940 to December 1941	401·27	183·24	·198	3·39	3·83	54·06	2·19	28·27
	Khurrianwala Distributary	5,000	May 1940 to December 1941	222·32	107·67	·221	2·62	3·03	41·06	2·07	27·01
	Shahkot Distributary	12,000	June 1940 to December 1941	168·19	89·74	·233	2·55	2·96	35·11	1·88	26·14
Tar-Khani	Lower Gugera Branch	272,500	May 1940 to December 1941	833·49	355·89	·194	4·87	5·61	73·14	2·34	26·73
	Lower Gugera Branch	259,000	May 1940 to December 1941	1,116·51	474·95	·181	5·14	5·73	92·35	2·36	26·77
Buchiana	Lower Gugera Branch	11,000	May 1940 to December 1941	1,875·49	710·57	·229	5·65	6·13	125·94	2·64	33·98
	Burala Branch	6,000	May 1940 to December 1941	1,572·28	600·48	·227	5·76	6·41	104·22	2·62	33·88
	Upper Gugera Branch	277,500	January 1941 to December 1941	3,654·35	1,179·23	·191	8·81	10·22	133·95	3·10	34·80

N. B.—The values of M × 10³ have been averaged up to July 1941 only.

TABLE

STATEMENT GIVING THE VALUES OF $P/\sqrt{Q}$, $R/Q^{\cdot 345}$ AND THEIR PERCENTAGE

Serial No.	Canal	Channel	R. D.	Period Observed	
				From	To
1	Lower Chenab Canal.	Hafizabad Distributary	3,050	October 1933 ..	January 1939
2	Ditto	Main Line Upper ..	137,600	January 1935..	February 1939
3	Ditto	Main Line Lower ..	149,000	October 1933 ..	February 1939
4	Ditto	Jhang Branch Lower ..	111,700	July 1936 ..	March 1939
5	Ditto	Dhaular Distributary ..	12,000	Ditto ..	Ditto
6	Ditto	Mian Ali Branch .	9,500	November 1934	April 1939
7	Ditto	Rakh Branch ..	7,260	Ditto	Ditto
8	Ditto	Jhang Branch .	7,260	Ditto	Ditto
9	Ditto	Main Line	181,000	January 1935..	March 1937
10	Ditto	Udhoki Distributary ..	2,500	May 1937 ..	April 1939
11	Ditto	Gugiana Distributary .	7,345	September 1933	March 1939
12	Ditto	Sarangwala Distributary	6,300	October 1933 ..	Ditto
13	Ditto	Jhang Branch ..	111,700	Ditto ..	Ditto
14	Ditto	Khai Distributary ..	4,943	Ditto ..	October 1934

N.B.—V denotes percentage variation

XXIII

DEVIATIONS FROM THE MEAN FOR 42 REGULARLY OBSERVED CHANNELS

$P_f \times \overline{Q}$	V of column 5	$B_f Q$ [illegible]	V of column 7	$A_f Q$ [illegible]	V of column 9	$\frac{S_{act}}{S_{cal}}$ in 10
2·494	−8·9	·465	+5·2	1·124	−2·0	·33/·35
3·067	+12·7	·410	−7·2	1·160	+1·7	·19/·15
3·039	+11·0	·47	−5·7	1·173	+2·3	·20/·17
2·677	−2·3	·430	−1·4	1·106	−3·6	·23/·18
2·813	+2·7	·396	−10·4	1·061	−7·5	·22/·20
2·917	+6·5	·392	−11·3	1·076	−6·2	·22/·22
2·743	+0·1	·424	−4·1	1·094	−4·6	·21/·22
2·710	−1·1	·443	+2·0	1·117	−2·6	·21/·19
3·102	+13·3	·408	−7·7	1·173	+2·3	·20/·21
2·770	+1·4	·419	−5·2	1·122	−2·2	·31/·30
3·024	+10·4	·420	−5·0	1·230	+7·2	·28/·29
3·164	+15·5	·396	−10·4	1·208	+5·3	·25/·30
2·823	+3·1	·436	−1·4	1·148	+0·1	·18/·17
2·666	−2·7	·442	0	1·138	−0·8	·29/·26

V from the mean of 42 channels.

TABLE

Serial No.	Canal	Channel	R. D.	Period Observed From	Period Observed To
15	Lower Chenab Canal.	Nasrana Distributary ..	3,275	October 1933 ..	October 1934
16	Ditto	Jamal Jatti ..	2,300	Ditto ..	Ditto
17	Lower Jhelum Canal.	Khunan Distributary ..	3,100	February 1936	August 1937
18	Ditto	Pindi Distributary ..	1,550	Ditto	Ditto
19	Ditto	Khatwan Distributary ..	5,020	Ditto	March 1937
20	Ditto	Northern Branch ..	227,000	Ditto	August 1937
21	Ditto	Ditto ..	210,100	Ditto	Ditto
22	Ditto	Ditto ..	204,200	Ditto	Ditto
23	Ditto	Burala Distributary ..	1,700	Ditto	Ditto
24	Ditto	Northern Branch Upper	272,600	December 1936	December 1937
25	Ditto	Northern Branch Lower	281,500	August 1936 ..	Ditto
26	Ditto	Sillanwali ..	7,000	July 1936 ..	Ditto
27	Ditto	N. B. U., S., Ludewala ..	163,000	September 1933	January 1936
28	Ditto	N. B. D., S., Ludewala ..	170,289	Ditto	Ditto

N.B.—V denotes percentage variation.

XXIII– continued

$P/\sqrt{Q}$	V of column 5	R Q	V of column 7	A,Q	V of column 9	$\frac{S^{1}_{act}}{S^{1}_{cal}}$ in 10
2·421	−11·6	·443	−2·0	·996	−13·2	·17 / ·20
3·084	+12·6	·412	−6·8	1·252	+9·2	·33 / ·29
2·789	+1·8	·431	−2·5	1·185	+3·3	·30 / ·28
2·419	−11·7	·473	+7·0	1·118	−2·5	·27 / ·28
2·738	+0·7	·429	−2·9	1·150	+0·3	·31 / ·25
2·622	−4·3	·455	+2·9	1·126	−1·8	·15 / ·18
2·491	−9·1	·478	+8·1	1·124	−2·0	·15 / ·18
2·671	−2·5	·463	+4·8	1·162	+1·3	·14 / ·17
2·466	−10·0	·466	+5·4	1·121	−2·3	·27 / ·28
2·613	−4·6	·473	+7·0	1·168	+1·8	·15 / ·17
2·747	+0·3	·445	+0·7	1·157	+0·9	·15 / ·17
2·580	+5·1	·438	−0·9	1·232	+7·4	·34 / ·30
2·637	−3·7	·455	+2·9	1·128	−1·7	·13 / ·17
2·679	−2·2	·450	+1·8	1·139	−0·7	·16 / ·16

from the mean of 42 channels.

TABLE

Serial No.	Canal	Channel	R. D.	Period Observed	
				From	To
29	Lower Jhelum Canal.	Bhek Distributary ..	1,500	September 1933	January 1936
30	Ditto	Sulki Branch ..	2,300	October 1935 ..	December 1935
31	Ditto	Northern Branch Lower	73,500	February 1935	June 1936
32	Ditto	Mitha Lak Distributary	4,000	Ditto	Ditto
33	Ditto	Fatehpur ..	10,200	Ditto	April 1936
34	Ditto	Southern Branch ..	102,500	September 1933	November 1933
35	Ditto	Kirana Distributary ..	2,675	October 1933 ..	December 1934
36	Ditto	Lalian Distributary ..	1,350	Ditto	November 1934
37	Upper Bari Doab Canal.	Aliwal Distributary ..	1,500	February 1934	February 1936
38	Lower Bari Doab Canal.	Main Line ..	115,000	March 1938 ..	November 1938
39	Ditto	Dhunniwala Distributary	18,500	Ditto ..	July 1938
40	Ditto	Jandwala Minor ..	3,050	Ditto ..	November 1938
41	Ditto	Main Line ..	93,400	Ditto ..	Ditto
42	Sirhind Canal Circle.	Akhara Distributary ..	9,050	May 1936 ..	June 1938

N.B.—V denotes percentage variat-

XXIII—concluded

P/ √Q	V of column 5	$R/Q^{\cdot 344}$	V of column 7	$A/Q^{\cdot 344}$	V of column 9	$\frac{S_{act}}{S_{cal}}$ in 10
2·765	−0 9	·433	−2 0	1·153	+0·5	·28 / ·28
2·987	+9 1	·385	−12·9	1·092	−4·8	·20 / ·22
2 690	+1·8	·453	+2 5	1·144	−0·3	·17 / ·18
2 194	−19·9	·518	+17·2	1·089	−5·1	·20 / ·24
2 574	−6 0	·486	+10·0	1 211	+5·6	·22 / ·27
2·915	+6 4	·449	+1·6	1 235	+7·7	·14 / ·17
2 878	+5 1	·421	−4 8	1 149	+0·2	·20 / ·20
2 379	−13 9	·497	−12 4	1 112	−3·1	·16 / ·20
2 927	+6·9	·395	−10·6	1·108	−3 4	·30 / ·34
2 830	+3·3	·475	+7·5	1·244	+8·5	·13 / ·12
2·809	+2·6	·456	+3·2	1·234	+7·6	·21 / ·24
2·652	−3·2	·462	+4·5	1·179	+2·8	·17 / ·26
2·806	+2·4	·476	+7·7	1·235	+7·7	·12 / ·12
2·347	−14·3	·451	+2·0	1·013	−11·7	·20 / ·25

from the mean of 42 channels.

TABLE

SHOWING THE AVERAGE VALUES OF THE DISCHARGE ELEMENTS FOR

Serial No.	Canal	Site	Name of Channel	Period Observed	
				From	To
1	Lower Chenab Canal.	Sagar ..	Hafizabad Distributary R. D. 3 050.	October 1933 .	January 1939
2	Ditto	Do ..	M. L. U. R. D 137 600.	January 1935.	February 1939
3	Ditto	Do. ..	M L L R. D. 149 000	October 1933 ..	Ditto
4	Ditto	Tawan .	J B. L R. D 111,700.	July 1936 .	March 1939
5	Ditto	Do .	Dhaular Distributary 12,000	Ditto ..	Ditto
6	Ditto	Nanuana ..	Mian Ali Branch 11/34 to 3/37, 2,850/9,500 (4/37 to 4/39)	November 1934	April 1939
7	Ditto	Do. ..	Rakh Branch 7,260.	Ditto	Ditto
8	Ditto	Do. .	Jhang Branch 7,260	Ditto	Ditto
9	Ditto	Do. ..	Main Line 181,000 5/37 to 10/37.	January 1935 ..	March 1937
10	Ditto	Do. ..	Udhoki Distributary 2 494/2,372 (11/37 to 4/39).	May 1937 ..	April 1939
11	Ditto	Kot Khuda-yar.	Gagiara Distributary 7,345	September 1938	March 1939
12	Ditto	Ditto	Sarangwala Distributary 6,300.	October 1938 ..	Ditto
13	Ditto	Ditto	Jhang Branch 221,000	Ditto ..	Ditto
14	Ditto	Ditto	Khai Distributary R. D. 4,943.	October 1934 ..	October 1934

XXIV

THE PERIOD THE CHANNELS WERE UNDER OBSERVATION IN THE PUNJAB

Q.	A.	S.	B.	P.	V.	A. M.	D.
22·50	16·05	·33	1·36	11·38	1·40	26·45	1·84
		·35	1·33	13·28			1·61
9,005·39	2,778·19	·19	9·48	292·99	3·25	42·02	10·34
		·15	9·78	265·72			9·41
4,463·36	1,534·77	·20	7·57	203·03	2·91	42·17	8·05
		·17	7·74	187·07			7·68
532·45	235·44	·23	1·80	61·76	2·27	27·04	4·35
		·18	3·81	64·60			4·17
289·60	134·26	·22	2·80	47·87	2·16	26·46	3·18
		·20	3·11	47·66			3·58
551·63	236·14	·22	3·46	68·51	2·34	33·78	3·83
		·22	3·85	65·77			4·26
1,080·90	426·48	·21	4·72	90·18	2·54	40·51	5·24
		·22	4·82	92·06			5·17
2,824·31	988·90	·21	6·87	144·02	2·86	42·53	7·64
		·19	6·64	148·79			6·70
4,419·80	1,521·58	·20	7·39	206·19	2·90	41·44	7·93
		·71	7·71	186·14			7·66
40·35	26·38	·31	1·50	17·63	1·53	26·06	1·81
		·30	1·61	17·79			1·92
28·96	21·79	·28	1·34	16·27	1·33	23·17	1·62
		·29	1·44	15·07			1·80
59·41	39·52	·26	1·62	24·39	1·50	27·75	1·86
		·30	1·83	21·58			2·23
2,102·43	789·94	·18	6·10	129·42	2·66	33·83	6·68
		·17	6·02	128·38			6·45
40·03	26·59	·29	1·58	16·87	1·50	21·84	1·96
		·26	1·61	17·72			1·95

TABLE:

Serial No.	Canal	Site	Name of Channel	Period Observed — From	To
15	Lower Chenab Canal.	Kot Khudayar.	Nasrana Distributary 3,275	October 1933 ..	October 1934
16	Ditto	Ditto	Jamal Jatti Distributary 2,300.	Ditto ..	Ditto
17	Lower Jhelum Canal.	Rurala .	Khunan Distributary 3,100.	February 1936	August 1937.
18	Ditto	Do. .	Pindi Distributary 1,350	Ditto	Ditto
19	Ditto	Do .	Khatwan Distributary 5,020.	Ditto	March 1937
20	Ditto	Do .	N Branch R D 227,000.	Ditto	August 1937
21	Ditto	Do .	N. Branch R D. 210,100	Ditto	Ditto.
22	Ditto	Do	N. Branch R D 204,200.	Ditto	Ditto.
23	Ditto	Do ..	Rurala Distributary R D 1,700.	Ditto	Ditto
24	Ditto	Manguana ..	N. B U. R. D 272,600.	December 1936	December 1937.
25	Ditto	Do. .	N. B. L. R. D. 281,500	August 1936 ..	Ditto
26	Ditto	Do. ..	Sillanwali Distributary 7,000.	July 1936 ..	Ditto
27	Ditto	Ludewala ..	N. B. U/S. Ludewala 163,000.	September 1933	January 1936.
28	Ditto	Do. ..	N. B. D/S 170,289.	Ditto	Ditto

XXIV—continued

Q.	A.	S.	R.	P.	V.	A. M.	D.
272·75	119·76	·17	3·00	39·99	2·28	26·28	3·60
		·20	3·05	46·26			3·77
27·26	21·06	·33	1·29	16·10	1·31	22 15	1·54
		·29	1·41	14·62			1·69
5·79	5·31	·30	79	6 71	1·09	15·05	1·16
		·28	·84	6·71			1·14
15·95	11·90	·27	1·23	9 66	1·34	18 58	1·78
		·28	1·18	11·18			1·55
27·18	19·29	·31	1·34	14·38	1·41	19·44	1 70
		25	1·41	14·60			1·72
611 42	269·76	15	4 16	64·83	2·27	27 04	4·71
		18	3·99	69·24			4·86
659·61	287·31	·15	4 49	63·97	2 30	28·58	5·14
		·18	4·09	71 90			4·96
735 75	326·11	·14	4·51	72·43	2·25	27·14	5·08
		·17	4·24	75 94			5·20
19·18	13·96	·27	1·29	10·80	1·37	19·87	1·80
		·28	1 26	12 26			1·63
544·26	253·37	·15	4 16	60·96	2 14	24·78	4·76
		17	3·84	65·32			4·70
493·00	230·75	·15	3·78	60·09	2 14	25·12	4·26
		·17	3·71	62·16			4·58
7·81	7·13	·34	·89	8·05	1 09	17·11	1·21
		30	·93	7 83			1·20
1,188·65	476·98	·13	5·24	90·94	2·49	31·12	5·84
		·17	4·98	96·54			6·04
768·23	331·75	·16	4·45	74·26	2·32	30·35	5·01
		·18	4·30	77·62			5·07

TABLE

Serial No	Canal	Site	Name of Channel	Period Observed	
				From	To
29	Lower Jhelum Canal.	Ladewala ..	Black Distributary 1,500.	September 1933	January 1936..
30	Ditto	Do.	Sulki Branch 2,300.	October 1933 ..	December 1935
31	Ditto	Satghara ..	N. B. L. R. D. 73,500.	February 1935	June 1936
32	Ditto	Do. ..	Mithi Lak Distributary 4,000	Ditto ..	Ditto
33	Ditto	Do ..	Fatehpur Distributary 10,200.	Ditto ..	April 1936
34	Ditto	Gullapur ..	Southern Branch 102,500.	September 1933	November 1934
35	Ditto	Do. ..	Kuana Distributary 2,675.	October 1933 ..	December 1934
36	Ditto	Do ..	Lahan Distributary 1,700.	Ditto	November 1934
37	Upper Bari Doab Canal.	Aliwal ..	Aliwal Distributary 1,500.	February 1934	February 1936
38	Lower Bari Doab Canal.	Khokhar ..	M. L. R. D. 115,000.	March 1938 ..	November 1938
39	Ditto	Do. ..	Dhunniwala Distributary 18,500.	Ditto ..	July 1938
40	Ditto	Do. ..	Jandeala Minor 3,050.	Ditto ..	November 1938
41	Ditto	Do. ..	M. L. R. D. 93,400.	Ditto ..	Ditto
42	Sirhind Canal..	Akhara ..	Akhara Distributary 9,050.	May 1936 ..	June 1938

XXIV—concluded

Q.	A.	S.	R.	P.	Y.	A. M.	D.
51·48	35·59	·28	1·73	20·59	1·56	25·62	2·09
		28	1·79	20·85			2·15
354·48	164·27	·20	2·92	56·25	2·16	30·04	3·21
		·22	3·33	52·72			3·88
1,407·05	558·66	·17	5·52	100·92	2·52	35·20	6·09
		·18	5·27	105·03			6·95
133·19	71·03	·20	2·80	25·32	1·85	27·29	3·74
		·24	2·40	32·31			2·98
44·61	31·04	·22	1·80	17·19	1·43	23·29	2·35
		·27	1·67	18·70			2·16
784·51	366·06	·14	4·48	81·64	2·15	27·43	5·02
		·17	4·33	78·43			5·29
440·33	207·92	·20	3·44	60·38	2·12	28·53	3·85
		·20	3·58	58·74			4·11
361·48	107·09	·16	3·79	44·85	2·12	27·74	4·57
		·20	3·35	53·23			4·14
80·14	46·81	·30	1·79	26·20	1·72	35·67	2·06
		·34	2·03	25·07			2·33
5,676·92	1,908·92	·13	9·37	213·24	2·84	29·64	10·27
		·12	8·38	210·98			9·21
42·32	30·22	·21	1·66	18·27	1·40	19·87	2·04
		·24	1·64	18·21			2·16
99·41	59·80	·17	2·25	26·44	1·66	23·74	2·74
		·26	2·18	27·92			2·87
5,810·25	2,024·00	·12	9·46	213·87	2·87	30·34	10·34
		·12	8·45	213·42			9·47
17·92	11·91	·20	1·22	9·70	1·51	17·77	1·78
		·26	1·23	11·57			1·57

TABLE

STATEMENT SHOWING EFFECT OF CILL LEVEL OF

Date / Time		Regulator 1	2	3	4	5	6
11-7-41	Cill level of gates	710·7	710·7	710·7	710·3	709·6	709·6
8 a. m.	Coarse silt entering canal.	0·022	0·031	0·056	0·308	0·420	0·196
					Average silt 0·134 grams per		
11-7-41	Cill level of gates	710·5	710·5	710·7	710·8	710·8	710·7
1 p. m.	Coarse silt entering canal.	0·028	0·042	0·056	0·084	0·112	0·056
					Average 0·058 grams per		
14-7-41	Cill level of gates	710·3	710·3	710·6	710·7	710·7	710·3
7 a. m.	Coarse silt entering canal.	0·014	0·021	0·056	0·070	0·084	0·084
					Average 0·045 grams per		
18-7-41	Cill level of gates	711·7	711·7	711·8	712·3	712·3	712·3
7 a. m.	Coarse silt entering canal.	0·014	0·028	0·042	0·014	0·011	0·011
					Average 0·017 grams per		
22-7-41	Cill level of gates	711·6	711·6	711·6	711·0	711·0	711·0
9-30 a.m.	Coarse silt entering canal.	0·028	0·084	0·056	0·084	0·070	0·042
					Average 0·049 grams per		
26-7-41	Cill level of gates	710·9	710·8	710·7	710·6	710·5	710·4
9-45 a.m.	Coarse silt entering canal.	0·011	0·011	0·014	0·014	0·011	0·011
					Average 0·011 grams per		
31-7-41	Cill level of gates	709·9	709·8	709·7	709·6	709·5	709·4
9-15 a.m.	Coarse silt entering canal.	0·011	0·011	0·017	0·017	0·014	0·011
					Average 0·012 grams per		
31-7-41	Cill level of gates	709·4	709·4	709·4	709·5	709·5	709·5
1-30 p.m.	Coarse silt entering canal.	0·017	0·021	0·025	0·014	0·014	0·008
					Average silt 0·014 grams		

XXV

GATES ON SILT ENTERING CANAL

Bays 7	Bays 8	Coarse silt entering pocket	Coarse silt upstream river line 13-14	Pond Level	Average silt in pocket 100—600	Maximum depth of silt in pocket	Canal silt / Pocket silt	Difference in sill level
								Feet
709·6 0·028 litre	709·6 0·008	0·140	0·168	713·05	8·1	712·8	1 : 1	1·1
708·9 0·056 litre	708·9 0·028	0·140	0·168	713·15	8·1	712·8	1 · 2·5	1·6
709·4 0·021 litre	709·4 0·011	0·050	0·070	713·25	8·7	711·9	1 : 1	·9
710·9 0·008 litre	710·9 0·006	0·031	0·840	714·8	8·8	712·3	1 : 2	·8
710·2 0·014 litre	710·2 0·014	0·196	0·420	714·15	9·4	712·4	1 : 4	1·4
710·3 0·010 litre	710·3 0·008	0·014	0·224	713·52	2·3	708·3	1 : 1	·6
709·3 0·011 litre	709·3 0·008	0·059	0·168	712·5	3·5	708·85	1 : 5	
709·5 0·007 per litre	709·5 0·006	0·059	0·168	712·4	3·5	708·85	1 : 4	·1

TABLI

STATEMENT SHOWING EFFECT OF CILL LEVEL OI

Date / Time		Regulatoi					
		1	2	3	4	5	6
11-7-41	Cill level of gates	710·7	710·7	710·7	710·3	709·6	709·6
8 a. m.	Coarse silt entering canal.	0·022	0·031	0·056	0·308	0·420	0·196
					Average silt 0·134 grams per		
11-7-41	Cill level of gates	710·5	710·5	710·7	710·8	710·8	710·5
1 p. m.	Coarse silt entering canal.	0·028	0·042	0·056	0·084	0·112	0·056
					Average 0·058 grams per		
14-7-41	Cill level of gates	710·3	710·3	710·6	710·7	710·7	710·3
7 a. m.	Coarse silt entering canal.	0·014	0·021	0·056	0·070	0·084	0·084
					Average 0·045 grams per		
18-7-41	Cill level of gates	711·7	711·7	711·8	712·3	712·3	712·3
7 a. m.	Coarse silt entering canal.	0·014	0·028	0·042	0·014	0·011	0·011
					Average 0·017 grams per		
22-7-41	Cill level of gates	711·6	711·6	711·6	711·0	711·0	711·0
9-30 a.m.	Coarse silt entering canal.	0·028	0·084	0·056	0·084	0·070	0·042
					Average 0·049 grams per		
26-7-41	Cill level of gates	710·9	710·8	710·7	710·6	710·5	710·4
9.45 a.m.	Coarse silt entering canal.	0·011	0·011	0·014	0·014	0·011	0·011
					Average 0·011 grams per		
31-7-41	Cill level of gates	709·9	709·8	709·7	709·6	709·5	709·4
9.15 a.m.	Coarse silt entering canal.	0·011	0·011	0·017	0·017	0·014	0·011
					Average 0·012 grams per		
31-7-41	Cill level of gates	709·4	709·4	709·4	709·5	709·5	709·5
1-30 p m.	Coarse silt entering canal.	0·017	0·021	0·025	0·014	0·014	0·008
					Average silt 0·014 grams		

XXVI.

OBSERVATIONS AT TRIMMU

SLUICE

Tunnel Discharge	Coarse Silt			Non-Tunnel Pocket	Canal Discharge / Tunnel Discharge	E%
	Tunnel Pocket	Tunnel	Canal	Coarse		Coarse
4,934	·0110	·0117	·0104	·0153	1·140	5·5
5,066	·0128	·0159	·0101	·0159	1·128	21·4
5,054	·0105	·0134	·0078	·0150	1·051	26·0
5,094	·0158	·0189	·0130	·0175	1·101	17·8
5,124	·0097	·0123	·0073	·0098	1·095	24·67
4,963	·0125	·0163	·0086	·0123	·976	31·26
5,124	·0077	·0094	·0060	·0078	1·037	21·74
5,027	·0097	·0117	·0078	·0140	1·057	19·65
1,152	·0164	·0182	·0806	·0390	4·913	2·1
7,388	·0140	·0182	·0086	·0546	·779	38·6
7,178	·0335	·0548	·0070	·0910	·802	79·1
7,392	·0252	·0377	·0091	·0511	·779	63·9
7,242	·0200	·0286	·0091	·0520	·795	54·4
7,158	·0198	·0273	·0104	·0322	·804	47·3
7,251	·0099	·0101	·0096	·0110	·767	28·0
7,320	·0184	·0227	·0127	·0247	·760	30·9
7,361	·0149	·0162	·0130	·0188	·702	12·7
7,302	·0112	·0120	·0086	·0130	·708	23·05
7,376	·0061	·0065	·0055	·0094	·701	9·7
7,301	·0073	·0075	·0070	·0058	·733	3·8
7,272	·0068	·0071	·0065	·0071	·736	4·9
7,265	·0041	·0045	·0036	·0039	·737	12·6
6,233	·0039	·0039	·0039	·0039	·859	Nil
6,302	·0057	·0068	·0044	·0045	·850	22·7
6,430	·0074	·0099	·0049	·0089	·833	33·5
6,404	·0081	·0110	·0078	·0104	·836	4·3

TABLE.

DATA FOR SILT AND ALLIED-

LEFT UNDER

Date	River Discharge upstream	Pond Level 480+	Upstream Discharge Non-Tunnel Pocket	Upstream Discharge Tunnel Pocket	Canal Discharge
June 1941—					
26th	59,855	10·5	5,998	10,571	5,632
27th	64,358	10·5	6,106	10,779	5,713
28th	63,802	10·5	6 092	10,365	5,311
30th	88,377	10·5	6,277	10,703	5,609
July 1941—					
2nd	79,032	11·0	6,137	10,733	5,609
3rd	80,880	10·0	5,900	9,805	4,842
4th	82,553	11·0	6,132	10,436	5,312
5th	96,250	11·0	6,188	10,339	5,312
7th	102,572	11·0	11,253	6,812	5,660
10th	131,751	10·0	12,391	13,145	5,757
11th	100,547	10·0	12,177	12,935	5,757
12th	91,402	10·0	9,271	13,149	5,757
14th	110,727	10·0	8,912	12,999	5,757
16th	76,486	10·0	8,819	12,915	5,757
18th	60,003	10·0	8,384	12,813	5,562
19th	59,867	10·0	8,333	12,882	5,562
21st	78,193	10·0	8,098	12,528	5,167
22nd	64,639	10·0	8,187	12,469	5,167
23rd	64,300	10·5	8,198	12,543	5,167
24th	59,750	10·5	8,010	12,656	5,355
25th	48,603	10·5	8,015	12,627	5,355
26th	39,855	10·5	5,888	12,620	5,355
28th	47,494	10·5	4,114	11,588	5,355
29th	54,468	10·5	4,242	11,657	5,355
30th	56,669	10·5	4,242	11,785	5,355
31st	91,459	10·5	6,084	13,759	5,355

XXVI.

OBSERVATIONS AT TRIMMU

SLUICE

Tunnel Discharge	Coarse Silt			Non-Tunnel Pocket	Canal Discharge / Tunnel Discharge	E%
	Tunnel Pocket	Tunnel	Canal	Coarse		Coarse
4,939	·0110	0117	·0104	·0153	1·140	5·5
5,063	·0128	·0159	·0101	·0159	1·128	21·4
5,054	·0105	0134	·0078	·0150	1·051	26·0
5,094	·0158	0189	·0130	·0175	1·101	17·8
5,124	·0097	·0123	0073	·0098	1·095	24·67
4,963	0125	·0163	·0086	·0123	·976	31·26
5,124	·0077	·0094	·0060	·0078	1·037	21·74
5,027	·0097	·0117	·0078	·0140	1·057	19·65
1,152	·0164	·0182	·0806	·0390	4·913	2·1
7,388	·0140	·0182	·0086	·0546	·779	38·6
7,178	·0335	·0548	·0070	·0910	·802	79·1
7,302	·0252	·0377	·0091	·0511	·779	63 9
7,242	·0200	·0286	·0091	·0520	·795	54·4
7,158	0198	·0273	·0104	·0322	·804	47·3
7,251	·0099	·0101	·0096	·0110	·767	28·0
7,320	0184	·0227	·0127	·0247	·760	30·9
7,361	0149	0162	·0130	·0188	·702	12·7
7,302	0112	0120	·0086	·0130	708	23·05
7,376	·0061	·0065	·0055	·0094	·701	9·7
7,301	·0073	·0075	·0070	·0058	·733	3·8
7,272	·0068	·0071	0065	·0071	·736	4·9
7,265	·0041	·0045	·0036	·0039	·737	12·6
6,233	0039	·0039	·0039	·0039	·859	Nil
6,302	·0057	·0068	·0044	·0045	·850	22·7
6,430	·0074	·0099	·0049	·0089	·833	33·5
6,404	·0081	·0110	·0078	·0104	·836	4·3

TABLE.

Date	River Discharge upstream	Pond Level 480+	Upstream Discharge: Non-Tunnel Pocket	Upstream Discharge: Tunnel Pocket	Canal Discharge
August 1941—					
1st	105,604	10·5	6,359	11,800	5,355
2nd	73,980	10·0	8,708	12,801	5,659
4th	57,342	10·0	7,074	12,960	5,659
5th	57,407	10·0	7,074	12,998	5,697
6th	56,963	10·0	6,877	12,918	5,697
7th	92,088	10·0	7,939	13,110	5,697
8th	124,210	10·0	9,789	12,909	5,697
9th	105,765	10·0	12,062	12,989	5,697
11th	74,276	10·0	8,152	10,711	4,700
12th	113,670	10·0	8,093	10,722	4,700
13th	85,129	10·0	7,910	10,708	4,760
14th	74,364	10·0	7,986	10,749	4,700
16th	60,376	10·0	7,420	10,739	4,700
18th	65,796	10·5	7,102	12,502	5,645
19th	92,987	10·7	7,126	12,552	5,645
20th	97,745	10·9	7,154	12,059	5,645
21st	110,798	11·0	7,115	12,047	5,645
22nd	98,964	11·1	9,120	7,631	1,200
23rd	72,608	11·2	8,065	7,543	1,200
25th	96,835	11·4	8,152	2,725	1,200
26th	67,200	11·5	8,136	3,265	1,330
27th	56,780	11·6	7,893	3,268	1,330
28th	57,773	11·7	8,007	3,266	1,330
September 1941—					
1st	41,249	12·0	5,877	11,372	5,694
2nd	41,249	12·0	5,877	11,372	5,694
3rd	53,867	12·0	5,910	11,313	5,634
4th	86,158	12·0	6,092	11,311	5,634
5th	82,148	12·0	7,053	10,126	4,749
6th	40,122	12·0	5,934	9,329	4,749

XXVI—CONCLUDED.

Tunnel Discharge	Coarse Silt			Non-tunnel Pocket	Canal Discharge / Tunnel Discharge	E%
	Tunnel Pocket	Tunnel	Canal	Coarse		Coarse
6,445	·0109	·0163	·0057	·0182	·831	47·0
7,142	·0129	·0153	·0099	·0202	·792	23·4
7,301	·0084	·0101	·0062	·0156	·775	26·0
7,301	·0098	·0117	·0073	·0169	·780	25·4
7,221	·0088	·0110	·0060	·0130	·759	31·8
7,413	0180	0234	·0109	·0312	·769	39·3
7,212	·0200	·0263	·0120	·0254	·790	39·9
7,292	·0165	·0208	·0109	·0250	·781	33·7
6,011	·0117	·0143	·0083	·0156	·782	28·0
6,022	·0164	·0188	·0133	·0221	·780	18·9
6,008	·0141	·0176	·0096	·0189	·792	31·8
6,049	·0114	0156	·0060	·0156	·777	47·4
6,039	·0088	·0114	·0055	0158	·778	37·6
6,857	·0111	·0156	·0057	·0117	·823	48·8
6,907	0149	·0211	·0073	·0182	·817	51·0
6,414	·0106	·0150	0055	·0188	·880	47·9
6,402	·0114	·0169	·0052	·0195	·882	54·4
6,431	0116	0130	·0039	0195	·787	66·2
6,343	0051	0055	0029	0133	·789	42·9
1,525	0026	0026	·0026	·0016	787	*Nil*
1,935	0026	0026	0026	·0010	687	*Nil*
1,938	0014	0052	0031	·0075	686	28·9
1,936	·0010	·0052	·0021	·0055	687	42·9
5,678	·0060	·0068	·0052	·0068	1·003	13·3
5,678	·0065	·0068	·0062	·0052	1·003	4·6
5,670	·0048	·0052	·0044	·0052	·992	8·3
5,077	·0067	·0072	[illegible]	·0127	[illegible]	[illegible]
5,077	·0063	·0072	[illegible]	[illegible]	[illegible]	[illegible]
4,780	·0044	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

TABLE XXVII

SILT AND ALLIED OBSERVATIONS AT TRIMMU HEADWORKS (RIGHT UNDERSLUICES)

Date		River Discharge Upstream	Pond Level 480+	Discharge		Canal	Coarse Silt			E%	Canal Discharge
				Pocket	Under-sluice		Pocket	Under-sluice	Canal		Undersluice Discharge
August 1941—											
1st	..	103,601	10·0	9,875	8,325	1,550	·0187	·0186	·0191	N	·186
2nd	.	73,980	10·1	11,611	10,061	1,550	·0141	·0147	·0104	26·2	·154
4th	..	57,342	10 2	10,534	8,883	1,651	0063	·0067	·0043	31·7	·186
5th	..	57,407	10·2	10,561	8,883	1,678	·0063	·0067	·0043	31·7	·189
6th	..	56,963	10·2	10,254	8,576	1,678	·0065	·0072	·0030	53·8	·196
7th	..	92,088	10·2	11,293	9,588	1,705	·0133	·0141	·0091	31·0	·178
8th	..	124,210	10·2	12,609	10,904	1,705	·0148	·0152	·0121	18·2	·156
9th	..	105,765	10·1	12,406	10,701	1,705	·0126	·0130	·0100	20·6	·159
11th	..	74,276	10 1	10,764	9,059	1,705	·0078	·0082	0056	28·2	·158
12th	..	113,670	10·0	10,672	9,070	1,602	·0125	·0134	·0074	40·8	·177
13th	..	85,129	10·1	10,676	9,025	1,651	·0105	·0113	·0061	41·9	·183
14th	..	74,364	10·1	10,526	8,848	1,678	·0078	0082	·0056	28·2	·190
16th	.	60,376	10·2	9,849	8,088	1,761	·0080	·0091	·0030	62·5	·218
18th	..	65,746	10·7	10,159	8,371	1,788	·0073	·0090	·0043	41·1	·214
19th	..	92,987	10 9	11,116	9,299	1,817	·0105	0111	·0074	29·5	·195
20th	..	97,745	11·1	10,842	9,025	1,817	·0066	·0071	·0039	40·9	·201
21st	..	110,798	11·3	10,186	8,369	1,817	·0126	·0130	·0108	14·3	·217
22nd	..	98,964	10 9	11,850	10,033	1,817	·0089	·0098	·0039	56·2	·181
23rd	..	72,608	11·0	10,785	8,968	1,817	·0037	·0039	0026	29·7	·203
25th	..	96,835	11·3	10,945	9,173	1,772	·0026	·0026	·0026	Zero	·193
26th	..	67,260	11·3	10,756	8,984	1,772	·0026	·0026	·0026	,,	·197
27th	..	56,780	11·4	..	Closed	Closed	..	Closed	Closed	C	C
28th	..	57,773	11·6	..	,,	,,	..	,,	,,	,,	,,

TABLE XXVIII

STATEMENT SHOWING THE MEAN DIAMETER (mm) OF BED SILTS TAKEN FROM THE LEFT AND RIGHT POCKETS AT TRIMMU HEADWORKS AND HAVELI MAIN LINE AND RANGPUR CANAL

Date	Left Pocket		Right Pocket		Haveli Main Line R. D. 1,000	Rangpur Canal R. D. 2,500
	Bay 3	Bay 4	Bay 1	Bay 2		
May 1941—						
20th	·1775		3818	2947	2950	·3572
21st ..	·2792		3471	3000	·2766	·3340
22nd	2861	.	·3486	3107	2440	·3347
23rd	[illegible]		3657	3103	2577	·3064
24th	2372		3852	3128	·2523	3082
26th	2510	.	3540	3325	2492	·3551
27th	·2765		3834	3211	·2469	3396
28th	·2702		·3552	3447	2468	·3421
29th	2517		3340	2949	2374	3483
31st .	2480	.	·3761	·3245	·2739	3549
June 1941—						
4th .	2446	..	3510	·3073	.	·3590
5th	·2[illegible]		3434	3074		·3055
6th ..	·2361			..		
7th .	·3099	..	.	.		
9th .	2627	..		..		.
10th .. .	·2677	..	·3367	3319		·3109
11th .		.	2642	3196	..	·3079
13th . .	·2577		·3054	·3107	3489	·3422
15th ..	·2899	.	·3271	·3215	2130	·3367
16th	..	.	·3208	·2872	..	·3229
23rd	·3150	..	·3501	.	·1985	·2973
24th	·3256	..	·3242	..	·1629	·3085
25th	·3158	..	·3365	..	·1825	·3003
26th	·3245	..	·3430	..	·1783	3038
27th	·3484	..	·2973	..	..	·3073
28th	·3226	..	·3165	..	..	·2981
30th	·3018	..	·3381	..	..	·3056

TABLE XXXII

STATEMENT SHOWING THE VALUE OF MEAN DIAMETER OF BED SILTS TAKEN FROM SIDHNAI LEFT POCKET—BAY No. 5

Date	A.M.	Date	A.M.	Date	A.M.
10th May 1941 ..	·1593	19th June 1941 ..	·1683	25th August 1941 ..	·0981
12th May 1941 ..	·1893	20th June 1941 ..	·1959	27th August 1941 ..	·1265
13th May 1941 ..	·1748	23rd June 1941 ..	·1541	29th August 1941 ..	·1282
14th May 1941 ..	·1671	24th June 1941 ..	·1038	1st September 1941	·1742
15th May 1941 ..	·1449	26th June 1941 ..	·0939	3rd September 1941	·1279
16th May 1941 ..	·1584	28th June 1941 ..	·1524	5th September 1941	·1292
19th May 1941 ..	·1807	30th June 1941 ..	·1704	10th September 1941	·1010
20th May 1941 ..	·1906	2nd July 1941 ..	·2010	12th September 1941	·1467
21st May 1941 ..	·1675	4th July 1941 ..	·2061	16th September 1941	·1120
22nd May 1941 ..	·1969	7th July 1941 ..	·1651	18th September 1941	·1385
23rd May 1941 ..	·1709	9th July 1941 ..	·1542	22nd September 1941	·1187
24th May 1941 ..	·1698	11th July 1941 ..	·1656	25th September 1941	·0992
26th May 1941 ..	·1736	14th July 1941 ..	·1470	27th September 1941	·1042
28th May 1941 ..	·1777	16th July 1941 ..	·1713	29th September 1941	·0768
29th May 1941 ..	·1852	21st July 1941 ..	·1573	1st October 1941 ..	·0736
30th May 1941 ..	·1868	24th July 1941 ..	·1621	3rd October 1941 ..	·0941
2nd June 1941 ..	·1868	26th July 1941 ..	·1536	7th October 1941 ..	·1466
3rd June 1941 ..	·1643	28th July 1941 ..	·1633	9th October 1941 ..	·1484
5th June 1941 ..	·1490	30th July 1941 ..	·1526	11th October 1941 ..	·0710
6th June 1941 ..	·1527	2nd August 1941 ..	·1477	15th October 1941	·1257
7th June 1941 ..	·1428	4th August 1941 ..	·1518	17th October 1941 .	·1180
9th June 1941 ..	·1760	6th August 1941 .	·1573	19th October 1941 .	·0996
10th June 1941 ..	·1601	11th August 1941 ..	·0674	21st October 1941 ..	·1580
11th June 1941 ..	·1823	13th August 1941 .	·0960	23rd October 1941 ..	·0902
14th June 1941 ..	·1934	15th August 1941 ..	·1168	26th October 1941 .	·1530
16th June 1941 ..	·1949	17th August 1941 ..	·1166	28th October 1941 ..	·1322
17th June 1941 ..	·1873	19th August 1941 ..	·1132	30th October 1941 ..	·1354
18th June 1941 ..	·1798	21st August 1941 ..	0848		

TABLE XXXIII.

TABLE

SILT AND ALLIED OBSERVATIONS

Date	River Discharge Upstream	Pond Level 460 +	Dis		
			Pocket	Excluder	Sidhnai
August 1941					
1st	4,062	5·00	3,869	Closed	3,679
2nd	4,062	5·00	3,920	Closed	3,730
4th	4,080	5·00	3,886	Closed	3,696
7th	4,062	5·00	3,800	93	3,607
11th	10,340	4·80	3,738	274	3,268
12th	11,003	4·30	3,557	233	3,128
13th	9,626	4·50	4,018	266	3,563
14th	11,285	4·50	4,087	260	3,638
16th	11,496	4·50	4,036	230	3,617
18th . . .	8,964	4·60	4,101	270	3,642
19th ..	9,662	4·80	4,171	270	3,705
20th	9,706	4·70	4,209	270	3,743
22nd	8,697	4·50	4,138	270	3,672
23rd	9,169	4·50	4,254	290	3,768
26th	13,199	4·80	4,252	230	3,729
27th	12,400	4·80	4,094	260	3,638
28th	8,341	4·80	3,916	Closed	3,720
29th	8,470	4·80	4,057	220	3,641
30th	7,761	4·80	3,987	170	3,621

XXXIII

...T SIDHNAI HEADWORKS

...ge		Coarse Silt in gms./litre				
...al Shah	Hithar	Pocket	Excluder	Sidhnai	Fazal Shah	Hithar
135	55	·0028	Closed	·0028	·0028	·0028
135	55	·0028	Closed	·0028	·0028	·0028
135	55	·0028	Closed	·0028	·0028	·0028
135	55	0051	0079	0049	0042	·0056
138	58	·0323	0450	0125	0090	·0140
138	58	·0317	·0520	0310	0110	·0250
131	58	·0256	·0600	·0270	0140	·0140
131	58	·0248	0580	0230	0140	·0140
131	58	·0172	·0330	0165	·0090	·0140
131	58	·0096	·0250	0085	0090	·0090
138	58	·0098	·0280	0085	·0090	·0090
138	58	·0097	·0270	·0085	·0090	0090
138	58	·0102	0340	·0085	0090	[illegible]
138	58	0150	·0470	·0128	·0090	[illegible]
138	58	·0333	·0830	·0205	0200	
138	58	·0248	·0430	·0240	0110	
138	58	·0112	Closed	0115	[illegible]	
138	58	·0121	·0350	·0110	[illegible]	[illegible]
138	58	·0076	·0210	·0070		

TABLE XXXIV

STATEMENT SHOWING THE MEAN DIAMETER, AND CALCULATED AND ACTUAL SLOPES OF—

Reach / R. D	Kharif Discharge 1940	Kharif Discharge 1941	Bed Silt Diameter 1940	Bed Silt Diameter 1941	Calculated Slope 1940	Calculated Slope 1941	Actual Slope 1940	Actual Slope 1941
	Cusecs	Cusecs	m.m.	m.m.	‰	‰	‰	‰
MAIN LINE UPPER								
0—40,000	10,636	10,322	·4132	·3889	·139	·133	·110	·156
40,000—76,000	10,158	10 217	·4308	·4332	·146	·146	·140	·139
76,000—120,000	9,864	9,923	·4164	·3802	·142	·132	·155	·158
120 000—140,000	9,255	9,737	·4116	·3407	·143	·120	·156	·167
MAIN LINE LOWER								
140,000—160,000	4,867	4,374	·4002	·3674	·160	·152	·22	·202
160,000—181,000	4,847	4,224	·3937	·3565	·158	·150	·20	·196
181,000—200,000	4,575	3,983	·3802	·3351	·154	·143	·19	·19

TABLE XXXV

STATEMENT SHOWING THE MEAN DIAMETER AND CALCULATED AND ACTUAL SLOPES OF UPPER GUGERA BRANCH

Reach / R. D.	Kharif Discharge		Bed Silt Diameter		Calculated Slope		Actual Slope	
	1910	1911	1910	1911	1910	1911	1910	1911
0— 21,000.. ..	5,103	5,162	·3714	·3660	·148	146	148	·144
21,000— 35,000.. ..	5,017	5,076	·3549	3581	142	144	151	..
40,000— 50,000..	4,976	5,035	·3603	·3570	145	144	155	·178
55,000— 65,000.. .	4,857	4,916	·3731	3661	·151	148	·159	·150
70,000— 90,000.. ..	4,830	4,889	3601	·3467	146	141	116	·164
95,000—101,000..	4,602	4,651	·3426	3354	·142	139	203	·180
102,000—115,000.. ..	4,537	4,596	·3320	·3212	·138	134	175	·175
120,000—161,000.	4,474	4,533	3338	·3323	140	138	153	·149
162,500—205,000 . ..	4,155	4,214	3386	·3456	·143	145	16	·150
210,000—213,000..	4,039	4,098	·3255	·3175	·139	·136	170	·879
215,000—218,500 . ..	3,936	3,995	·3218	·3063	·140	·133	·157	·637
225,000—240,000.. ..	3,799	3,858	·3329	·3252	·144	·140	·173	·170
245,000—250,000.. ..	3,743	3,802	·3146	·3270	·137	·142	·170	·170
252,000—260,000.. ..	3,743	3,802	·3177	·3082	·137	·135	·205	·174
270,000—281,500.. ..	3,672	3,731	·3013	·2990	·132	·131	·184	·204

TABLE XXXVI

STATEMENT SHOWING THE MEAN DIAMETER AND CALCULATED AND ACTUAL SLOPES OF LOWER GUGERA BRANCH

Reach / R. D.	Kharif Discharge		Bed Silt Diameter		Calculated Slope		Actual Slope	
	1940	1941	1940	1941	1940	1941	1940	1941
0— 15,000.. ..	1,870	1,936	·3095	·3146	·156	·157	·204	·216
28,000— 55,000.. ..	1,675	1,741	·2980	·2925	·155	·151	·23	·238
70,000— 81,500.. ..	1,533	1,619	·2907	·3074	·154	·161	·22	·220
86,000—103,200.. .	1,553	1,619	·2740	·2758	·146	·146	·199	·138
121,000—140,000.. ..	1,337	1,403	·2858	·2828	·157	·154	·21	·213
145,000—165,000.. ..	1,313	1,379	·2755	·2756	·152	·151	·212	·211
172,000—200,000.. ..	1,262	1,328	·2571	·2608	·145	·145	·21	·214
205,000—215,000.. ..	1,224	1,290	·2480	·2370	·141	·136	·208	·211
219,000—264,000.. ..	1,122	1,188	·2350	·2336	·138	·135	·19	·185
270,000—293,000.. .	823	889	·2247	·2257	·140	·140	·201	·191
295,000—323,000.. ..	616	672	·2231	·2216	·149	·146	·207	·200
327,000—387,000.. ..	527	593	·2080	·2074	·115	·142	·201	·213

TABLE XXXVII

STATEMENT SHOWING THE MEAN DIAMETER AND CALCULATED AND ACTUAL SLOPES OF BURALA BRANCH

Reach / R. D.	Kharif Discharge		Bed Silt Diameter		Calculated Slope		Actual Slope	
	1910	1911	1910	1911	1910	1911	1910	1911
0—30,000	1,664	1,626	2816	·2874	·148	·151	·160	·194
35,000—45,000 ..	1,650	1,612	2018	·2048	140	142	20	207
48,100—58,500	1,621	1,583	·2598	·2625	·139	141	193	204
59,500—76,500	1,530	1,492	·2638	·2647	·143	·144	21	197
77,500—100,000.. ..	1,479	1,441	2018	2600	·141	144	17	·265
110,000—114,900.. ..	1,407	1,369	·2359	·2387	·130	134	17	181
150,000—160,000.. ..	1,365	1,327	2245	2198	·128	126	16	·159
165,000—180,000.. ..	888	850	·2175	·2128	·134	·131	·185	·224
190,000—201,500.. ..	850	812	2136	·2107	·134	·134	·10	·209
205,500—220,000.. ..	822	784	2099	·1968	·133	·128	·21	·259
230,000—245,000.. .	571	533	·2148	·2099	·147	·146	·24	·295
249,300—282,300.. ..	508	470	·2038	·1850	·143	·135	·23	..
285,000—295,000.. ..	497	459	·194	·1901	·141	·130	·24	..
296,600—310,750.. ..	485	447	·191	·1828	·138	·134	·24	·234

TABLE XXXVIII

STATEMENT SHOWING MEAN DIAMETER CALCULATED AND ACTUAL SLOPES OF JHANG BRANCH UPPER

Reach / R. D.	Kharif Discharge		Bed Silt Diameter		Calculated Slope		Actual Slope	
	1910	1941	1940	1941	1940	1941	1910	1941
0— 36,500..	2,864	2,946	·3712	·3774	·167	·160	·211	·227
37,500— 68,100..	2,830	2,911	3641	3692	165	·166	·206	·210
69,100— 85,600	2,705	2,880	3526	3500	161	·160	·218	·216
90,000—125,000 ..	2,750	2,861	·3366	3317	·157	·152	..	·196
135,000—149,000	2,660	2,778	·3376	·3183	·158	·148	·181	·212
155,000—160,000	2,540	2,738	·3214	·3225	·152	·150	·161	·168
165,000—175,000	2,420	2,703	·3217	·3151	·154	·147	·175	·168
177,600—178,600 ..	2,420	2,625	·3133	·3093	·151	·146	..	·203
180,000—205,000.. ..	2,410	2,610	·3120	·3066	·153	·145	·166	·175
205,000—211,400.. ..	2,410	..	·3092	..	·153	..	·174	..
217,400—245,000 ..	2,096	2,238	·2971	·2923	·149	·144	·181	·179
248,500—257,000.. ..	..	1,874	.2823	·2831	.	·146	·186	·184
260,500—277,000.. ..	1,827	1,869	·2791	·2736	·144	·141	·165	·171
278,000—305,000.. ..	1,698	1,722	·2710	·2650	·143	·140	·170	·165

TABLE XXXIX

STATEMENT SHOWING THE MEAN DIAMETER CALCULATED AND ACTUAL SLOPES OF JHANG BRANCH LOWER

Reach R. D.	Kharif Discharge		Bed Silt Diameter		Calculated Slope		Actual Slope	
	1940	1941	1940	1941	1940	1941	1940	1941
0— 19,558.. ..	1,187	1,257	·2601	·2420	·153	·133	·182	·183
20,500— 36,500.. ..	1,107	1,165	·2606	·2370	·150	·138	·194	·189
37,500— 48,000.. ..	1,083	1,083	·2577	·2522	·149	·148	·174	·202
40,000— 70,500.. ..	988	1,028	·2562	·2509	·153	·140	·195	·184
71,500— 92,500.. ..	950	902	·2528	·2484	·152	·150	·183	·183
93,500—107,500.. ..	650	602	·2468	·2191	·160	·147	·215	·209
108,500—135,000.. ..	545	550	·2433	·2309	·164	·158	·191	·221
139,343—167,450.. ..	480	472	·2372	·2265	·166	·160	·217	·229
168,450—180,000.. ..	436	428	·2305	·2179	·164	·158	·193	·234

TABLE XLII

MONTHLY SURVEY OF WATER SURFACE SLOPES IN 1941—LOWER GUGERA BRANCH

Site	Slope in 1,000 24/4/41	Slope in 1,000 /5/41	Slope in 1,000 24/6/41	Slope in 1,000 19/9/41	Slope in 1,000 18/11/41
1,000 ..	..	..	..	..	..
5,000 ..	·184	..	·202	·222	·192
10,000 ..	·156	..	·229	·225	·227
15,000 ..	·215	..	·227	·229	·232
20,000 ..	·221	..	·225	·283	232
25,000 ..	·211	..	·265	·422	·426
30,000 ..	·211	..	·240	·239	..
35,000 ..	·221	..	·229	·229	·225
40,000 ..	·223	..	·216	·226	·224
45,000 ..	·219	..	·223	·223	·222
50,000 ..	·237	..	237	·232	·231
55,000 ..	·274	..	·280	·276	·273
60,000 ..	·220	..	·227	·240	·240
70,000 ..	..	..	..	·222	·119
75,000 ..	..	..	·220	·262	·261
80,000 ..	..	..	..	·338	·311
85,000 ..	..	..	..	..	·483
90,000 ..	..	..	..	..	·409
95,000 ..	..	..	·138	·151	·149
100,000 ..	..	..	..	..	·173
105,000	..	..	..	..	·220
110,000 ..	·200	..	·198	·209	·202
115,000 ..	·188	..	·197	·213	·206
125,000 ..	·218	..	·231	·238	·214
130,000 ..	·202	..	·204	·209	·210
135,000 ..	·210	..	·207	·210	·210
140,000 ..	·209	..	·210	·213	·209
145,000 ..	·212	..	·203	·226	·220

TABLE XLII—continued

Site	Slope in 1,000 24/4/41	Slope in 1,000 [illegible]/5/41	Slope in 1,000 24/6/41	Slope in 1,000 19/9/41	Slope in 1,000 18/11/41
150,000	·214	..	·208	·222	·219
155,000	·203	..	·205	·205	·202
160,000	·211	..	·212	·215	·212
165,000	·220	..	·226	·225	·227
170,000	·233	..	·251	..	·245
175,000	·221	..	·225	·220	·224
180,000	·201	..	·206	·203	·199
185,000	·214	..	·214	·215	·211
190,000	·226	..	·221	·223	·219
195,000	·210	..	·211	·211	·203
200,000	·192	..	·201	·213	·191
205,000	·204	..	·210	·223	·207
210,000	·211	..	·211	·205	·221
215,000	·209	..	·213	·192	·231
220,000	·202	..	·213	·378	·218
225,000	·188	..	·188	·189	·183
230,000	·185	..	·187	·184	·178
235,000	·190	..	·193	·188	·176
240,000	·192	..	·190	·187	·143
245,000	·181	..	·179	·177	·173
250,000	·187	..	·175	·175	·213
255,000	·191	..	·173	·173	·181
260,000	·162	..	·173	·154	..
275,000	·195	..	·201	·192	·185
280,000	·183	..	·182	·182	·187
285,000	·191	..	·189	·189	·221
290,000	·428	..	..	·416	..
300,000	·215	..	·211	·197	·246
305,000	·215	..	·207	·207	·198

TABLE XLII—CONCLUDED

Site	Slope in 1,000 24/4/41	Slope in 1,000 /5/41	Slope in 1,000 24/6/41	Slope in 1,000 19/9/41	Slope in 1,000 18/11/41
310,000 ..	·203	..	·203	·199	·199
315,000 ..	·112	..	·194	·174	·197
320,000 ..	·221	..	·230	·191	..
330,000 ..	·209	..	·309	·202	·200
335,000 ..	·200	..	·209	·210	·206
340,000 ..	·214	..	·219	·220	·215
345,000 ..	·213	..	·212	·215	·211
350,000 ..	·204	..	·206	·206	·206
355,000 ..	·192	..	·198	·195	·189
360,000 ..	·192	..	·192	·193	·191
365,000 ..	·215	..	·208	·203	·227
370,000 ..	·205	..	·206	·203	·247
375,000 ..	·114	..	·197	·205	·194
380,000 ..	·242	..	·221	·231	·217
385,000 ..	·211	..	·184	·227	·231

TABLE XLIII

RAKH BRANCH—MONTHLY SURVEY OF THE WATER SURFACE SLOPES

R. D.	4/41	5/41	7/41	9/41	11/41
5,000 ..	·222	..	..	·224	·220
10,000 ..	·201	·195	·201	·203	·196
15,000 ..	·190	·189	·198	·195	·178
20,000 ..	·208	·226	·204	·207	·208
30,000 ..	·206	·192	·185	·195	·196
35,000 ..	..	·211	·208	·220	·210
40,000 ..	..	·135	·123	·213	·130
45,000 ..	·190	..	·152	·163	·150
55,000 ..	·217	..	·205	·245	·220
60,000 ..	·258	..	·254	·273	·257
65,000 ..	·206	..	·205	·179	220
70,000 ..	187	·099	·196	·182	·207
75,000 ..	..	·183	·187	·173	·173
80,000 ..	·105	·190	·165	·157	·167
85,000 ..	..	..	·132	135	132
90,000 ..	..	..	·210	..	..
95,000 ..	·198	·209	·211	·206	200
100,000 .	·222	..	·227	..	..
105,000 ..	·189	·169	·185	190	·174
110,000 .	·134	..	·128	·140	..
113,500 ..	..	·287	..	..	..
115,000 ..	171	·213	·189	200	·182
120,000 ..	..	·205	·192	·194	·201
125,000 ..	..	..	·153	·166	..
130,000 ..	..	·182	·144	·146	..
135,000 ..	·175	..	·191	..	..
140,000 ..	·157	·155	·289	·183	·160
145,000 ..	·164	·171	·160	·167	·171
150,000 ..	·171	·173	·161	·154	·179

TABLE XLIII—CONCLUDED

R. D.		4/41	5/41	7/41	9/41	11/41
155,000	..	·161	·164	162	·146	·172
160,000	..	·145	·164	·145	· 62	·163
165,000	..	·155	·177	·133	·130	·160
175,000	..	·164	·172	·162	·160	·177
180,000	..	·169	·159	·153	·152	·169
185,000	..	·169	·167	·159	·150	·175
190,000	..	·172	·173	·167	·159	·187
195,000	..	·160	·151	·148	..	·148
200,000	..	·163	·157	·154	·162	·167
205,000	..	·163	·171	·170	·164	·182
210,000	..	·159	·167	·160	·152	·172
215,000	..	·184	·164	·156	·144	·16[illegible]
220,000	..	·151	·236	·153	·140	·165
225,000	..	·276	..	..	..	·303
230,000	..	..	..	·167	·272	·323
235,000	..	·232	·230	·226	·220	·243
240,000	..	·186	·188	·178	·160	·186
245,000	..	·111	·105	·095	..	·105
250,000	..	·161	·158	·164	·168	·181
255,000	..	·156	·155	·149	·156	·166
260,000	..	·159	·159	·157	·148	·157
265,000	..	·161	·149	·155	·130	·148
270,000	..	·171	·167	·167	·142	·169
275,000	..	·186	·204	..	·174	·197

TABLE XLIV

STATEMENT OF DISCHARGES OBSERVED AT R. D. 17,000, TARKHANI, WITH CURRENT METER AND VELOCITY ROD

	Discharge by Current Meter		Discharge by Velocity Rod		Mean Discharge		V. R.—C. M.	$\frac{\text{Difference}}{\text{C. M.}} \times 100$
	L to R	R to L	50 Ft.	100 Ft.	C. M.	V. R.	Difference	Variation per cent.
Set 1	194·21	189·75	192·88	191·27	191·98	192·08	+·10	+·05
Set 2	185·56	195·73	182·17	184·80	190·65	183·49	—7·16	—3·76
Set 3	181·47	183·37	196·62	191·93	182·42	194·28	+11·86	+6·50
Set 4	191·44	191·91	192·11	189·50	191·67	190·81	—·86	—·45
Set 5	197·67	197·06	187·85	185·52	197·37	186·69	—10·68	—5·41
Set 6	188·35	186·49	192·61	190·16	187·42	191·39	+3·97	+2·12
Set 7	193·71	190·21	186·64	184·62	191·96	185·63	—6·33	—3·30

TABLE XLV

STATEMENT OF DISCHARGES OBSERVED AT R. D. 3,116, HAFIZABAD DISTRIBUTARY WITH CURRENT METER AND VELOCITY ROD

	Discharge by Current Meter		Discharge by Velocity Rod		Mean Discharge		V.R.—C.M.	$\frac{\text{Difference}}{\text{C. M.}} \times 100$	
	L to R	R to L	50 Ft.	100 Ft.	C. M.	V. R.	Difference	Variation percentage	
Set 1 ..	20·37	19·66	22·02	21·78	20·01	21·90	1·89	9·44	
Set 2 ..	20·32	20·02	21·58	21·59	20·17	21·59	1·42	7·04	
Set 3 ..	20·20	19·93	21·92	21·82	20·06	21·87	1·81	9·02	
Set 4 ..	19·68	19·66	21·62	21·66	19·67	21·64	1·97	10·02	
Set 5 ..	19·80	20·26	21·22	21·53	20·03	21·37	1·34	6·69	
Set 6 ..	19·57	19·74	22·04	21·68	19·65	21·86	2·21	8·89	Length of rod used = 0·8 × total depth at pendant.
Set 7 ..	20·09	19·87	21·91	21·82	19·98	21·86	1·88	9·41	
Set 8 ..	20·05	19·94	22·39	22·02	19·99	22·20	2·21	11·06	
Set 9 ..	20·24	19·47	21·37	21·71	19·85	21·54	1·69	8·59	
Set 10 ..	19·98	19·85	21·63	21·79	19·91	21·71	1·80	9·04	
Set 1 ..	20·11	19·67	22·15	21·61	19·89	21·38	1·49	7·49	
Set 2 ..	19·89	20·05	21·69	21·59	19·97	21·64	1·67	8·36	
Set 3 ..	20·13	19·84	22·24	22·10	19·98	22·17	2·19	10·96	
Set 4 ..	20·60	20·22	21·70	21·68	20·41	21·69	1·28	6·32	Length of rod used = 0·9 × total depth at pendant.
Set 5 ..	20·31	20·52	21·54	21·44	20·41	21·49	1·08	5·02	
Set 6 ..	20·10	20·13	20·78	20·96	20·12	20·87	·75	3·78	

TABLE XLVI

STATEMENT OF DISCHARGE OBSERVATIONS AT R. D. 10,000, LAKHUANA DISTRIBUTARY, WITH CURRENT METER AND VELOCITY ROD

	Discharge by Current Meter		Discharge by Velocity Rod		Mean Discharge		V.R. — C.M.		
	L to R	R to L	50 Ft	100 Ft.	C. M.	V. R.	Difference	Variation percentage.	
Set 1	100·68	107·78	113·68	114·19	107·27	114·09	+6·66	+6·40	
Set 2	108·06	106·81	118·04	117·00	107·45	117·52	+10·07	+9·37	
Set 3	108·06	107·74	117·91	115·25	107·90	116·58	+8·68	+8·04	
Set 4	108·83	110·78	113·21	112·32	109·81	112·77	+2·96	+2·70	
Set 5	109·63	110·70	115·81	115·17	110·12	115·40	+5·37	+4·88	
Set 6	110·54	112·57	111·50	114·74	111·56	113·12	+1·56	+1·40	·8 D
Set 7	107·50	109·70	117·09	117·70	108·65	117·44	+8·79	+8·09	
Set 8	110·46	108·31	111·05	115·11	109·40	113·08	+3·68	+3·36	
Set 9	110·74	109·17	121·26	110·82	109·96	120·54	+10·58	+9·62	
Set 10	110·16	109·69	116·35	116·10	109·93	116·27	+6·34	+5·77	
Set 11	111·58	111·70	112·52	111·51	111·69	112·02	+0·33	+0·30	
Set 12	108·57	110·34	106·91	106·98	109·46	106·95	−2·51	−2·29	
Set 13	108·10	107·40	110·52	109·21	107·80	109·87	+2·07	+1·92	
Set 14	110·64	111·30	111·07	110·28	110·97	110·68	−0·29	− ·26	
Set 15	110·40	111·35	107·82	106·60	110·88	107·21	−3·67	−3·31	
Set 16 .. .	111·46	112·07	107·28	107·10	112·07	107·19	−4·88	−4·35	·9 D
Set 17	108·49	110·16	113·60	111·98	109·33	112·79	+3·46	+3·16	
Set 18	109·80	109·45	110·66	109·08	109·63	109·87	+0·24	+ ·22	
Set 19	110·36	109·02	112·16	110·87	103·14	111·52	+6·38	+6·07	
Set 20	110·30	109·93	113·40	111·55	110·12	112·48	+2·36	+2·14	

TABLE XLVII

STATEMENT OF DISCHARGE OBSERVATIONS AT R. D. 235,500, RAKH BRANCH, WITH CURRENT METER AND VELOCITY ROD

	Discharge by Current Meter		Discharge by Velocity Rod		Mean Discharge		V.R. —C.M.	Variation percentage.
	L to R	R to L	50 Ft	100 Ft.	C. M.	V. R.	Difference	
Set 1	402·94	403·14	411·76	406·38	403·04	409·07	+6·03	+1·50
Set 2	408·61	409·54	415·66	409·75	409·08	412·71	+3·63	+0·89
Set 3	393·34	400·49	415·05	413·15	396·92	414·10	+17·18	+4·33
Set 4	384·01	394·56	398·55	393·45	389·29	396·00	+6·71	+1·72
Set 5	406·50	408·42	420·07	363·92	407·46	396·99	—10·47	—2·57
Set 6	406·91	410·43	402·93	401·61	408·67	402·27	—6·40	—1·57 ·8 D
Set 7	416·92	421·46	439·07	438·54	419·19	438·81	+19·62	+4·68
Set 8	412·34	402·85	422·24	421·32	407·60	421·78	+14·18	+3·48
Set 9	415·82	415·53	415·85	422·23	415·68	419·04	+3·36	+0·81
Set 10	424·56	425·05	444·85	437·41	424·81	441·13	+16·32	+3·84
Set 11	429·72	425·81	426·09	420·90	427·77	423·50	—4·27	—1·00
Set 12	..	403·90	398·05	396·92	403·90	397·49	—6·41	—1·59
Set 13	410·02	416·32	410·18	401·15	413·17	405·97	—7·20	—1·71
Set 14	410·32	411·57	391·94	389·10	410·10	390·52	—19·58	—4·77
Set 15	399·86	419·76	402·49	410·94	409·81	406·72	—3·09	—0·75 ·9 D
Set 16	393·98	388·05	365·63	363·89	391·02	364·76	—26·26	—6·72
Set 17	383·69	379·21	371·16	371·12	381·45	371·14	—10·31	—2·70
Set 18	386·74	388·13	386·38	384·40	387·44	385·39	—2·05	—0·53
Set 19	398·77	402·21	410·87	407·81	400·49	409·34	+8·85	+2·21
Set 20	396·56	402·52	408·27	404·95	399·54	406·61	+7·07	+1·77

TABLE XLVIII

STATEMENT OF DISCHARGE OBSERVATIONS AT R. D. 11,000, L. G. BRANCH, WITH CURRENT METER AND VELOCITY ROD

	Discharge by Current Meter		Discharge by Velocity Rod		Mean Discharge		V.R.—C.M.	Variation percentage
	L to R	R to L	50 Ft.	100 Ft.	C. M	V. R.	Difference	
Set 1	1,689·33	1,691·01	1,756·02	1,563·21	1,690·19	1,659·54	—30·55	—1·81
Set 2	1,703·65	1,707·24	1,841·39	1,793·51	1,705·40	1,817·45	+112·05	+6·57
Set 3	1,718·08	1,711·57	1,797·52	1,770·94	1,714·83	1,784·23	+69·40	+4·05
Set 4	1,731·50	1,709·08	1,841·77	1,832·64	1,720·29	1,837·21	+116·92	+6·80
Set 5	1,698·36	1,779·75	1,852·21	1,869·80	1,739·06	1,861·01	+121·95	+7·01
Set 6	1,732·02	1,745·39	1,909·15	1,900·92	1,739·01	1,905·04	+166·03	+9·55 } 4 D
Set 7	1,767·16	1,758·40	1,917·63	1,912·95	1,762·78	1,915·29	+152·51	+8·65
Set 8	1,789·23	1,739·40	1,857·61	1,863·45	1,764·32	1,860·53	+96·21	+5·45
Set 9	1,820·43	1,791·41	1,938·19	1,928·94	1,805·92	1,933·57	+127·65	+7·07
Set 10	1,799·04	1,782·08	1,936·26	1,923·50	1,791·46	1,929·88	+138·42	+7·73
Set 11 ..	1,813·26	1,736·98	1,952·60	1,948·65	1,790·12	1,950·73	+160·61	+9·20
Set 12 .. .	1,853·40	1,774·27	1,764·00	1,887·49	1,813·84	1,825·75	+11·91	+0·66
Set 13 .. .	1,814·10	1,801·95	1,835·02	1,885·27	1,808·03	1,860·15	+52·12	+2·88
Set 14	1,853·33	1,875·93	1,935·05	1,831·64	1,864·63	1,833·80	—30·83	—1·65 } 9 D
Set 15	1,724·01	1,744·25	1,834·94	1,835·26	1,734·58	1,835·10	+100·52	+5·80
Set 16	1,712·69	1,710·63	1,773·20	1,777·43	1,711·66	1,775·32	+63·66	+3·72
Set 17	1,678·56	1,694·79	1,790·03	1,784·86	1,690·68	1,787·45	+100·77	+5·97

TABLE L

DISCHARGE FROM CANAL TO DRAIN

Canal bed level	No.	Canal level	Head in feet	Temperature in centigrade	Discharge in cc/sc	Q 20°
R. L. 5·0 ..	1	9·715	1·493	16°	8·9	9·90
	2		2·007	16°	12·2	13·55
	3		2·538	16°	15·4	17·10
	4		2·977	20°C, 16·5°C	18·6	20·22
	5		3·378	19·0°C, 16°C	21·4	23·78
	6		4·032	19·5°C, 16·0°C	25·5	28·37
	7		4·616	20·0°C, 16·0°C	27·6	30·67
	8		5·091	20·8°C, 16·9°C	31·3	33·66
	9		6·028	21·0°C, 17·5°C	37·3	40·10
R. L. 5·0 ..	1	9·188	1·062	20°C & 17·0°C	6·50	7·00
	2		1·656	19·5°C, 16°C	10·54	11·60
	3		2·270	20°C, 17°C	15·4	16·5
	4		2·735	20·5°C, 17·5°C	18·3	19·4
	5		3·216	19·5°C, 17·5°C	20·7	21·9
	6		3·672	19·0°C, 18°C	23·8	25·0
	7		4·114	20°C, 17·9°C	27·4	28·77
	8		4·634	21°C, 18·2°C	30·0	31·5
	9		5·434	22°C, 10°C	34·1	34·8
	10		5·984	21·2°C, 19·5°C	36·5	36·87
	11		6·521	22·0°C, 20·5°C	39·6	39·6
	12		6·924	22·0°C, 20·2°C	43·0	43·0
	13		7·654	22·1°C, 20·2°C	45·2	45·2
			K_{20° = ·00035 feet/sc.			
R. L. 5 0 ..	1	8·694	1·130	20·2°C	8·65	8·65
	2		1·649	20·0°C	12·6	12·6
	3		2·140	20·0°C	15·75	15·75
	4		2·635	20·5°C	19·7	19·5
	5		3·068	21·5°C	23·4	22·5
	6		3·530	20·5°C	25·4	25·1
	7		3·946	20·8°C	27·6	27·1
	8		4 400	21·5°C	30·7	29·7
	9		5·130	21·0°C	34·0	33·3
	10		5·633	21·8°C	36·4	34·7
	11		5·[illegible]88	21·5°C	39·4	37·5
	12		6·598	21·0°C	42·18	40·1
R. L. 6·0 ..	1	9·74	1·20	24·5°C	10·10	9·1
	2		2 06	24·5°C	17·15	15·5
	3		4·08	25·3°C	33·1	29·3
	4		5·02	25·0°C	39·5	35·3

TABLE LI

DISCHARGE FROM DRAIN INTO CANAL

Canal bed level	No.	Drain level	Head in feet	Canal temperature in centigrade	Discharge in c/s	[illegible]
R. L. 5·0 ..	1	9·769	0·570	23·7	7·0	
	2		1·256	24·2	9·0	
	3		1·700	24·0	[illegible]	
	4		1·993	24·0	[illegible]	
	5		2·749	24·1	[illegible]	
	6		3·368	24·1	[illegible]	
	7		4·435	20·1	[illegible]	

STATISTICAL SECTION

The investigations were carried out during the year under the following heads:—

(1) Analysis of Pressure Pipe Observations at Ferozepore Headworks.

(2) Pressure Pipe Observations for the Depressed Bays of Islam Weir.

(3) Design of Works on Clay or Sand—Clay Foundations.

(4) Gated Flow in Undersluice Models.

(5) Mean Velocity and Mean Silt Points.

(6) Sampler Vs. Sampler (Suspended Silt).

(7) Rolling Silt Observations at Tibri.

(8) Lacey's Basic Regime Theory.

(9) The Best Flow Formula.

(10) Design of Regime Channels.

(11) Difference between Shock and Deviation from Regime for a Channel in Alluvium.

(12) Rugosity Co-efficients on Lined Channels.

(13) Silt Entry into Canals as affected by the Supply and the Regulation at the Headworks.

(14) Silt Surveys.

(15) Relation between Clay per cent of a Soil and its Capacity for Absorbing Water.

(16) Effect of Soil Factors on Yield of Wheat.

(17) Changes in Thur Area in 8 Districts of the Punjab.

(18) Factors Affecting Area Irrigated.

(19) Lay-out of an Experiment at Jaranwala Farm.

Besides the above, the following which were in the nature of advisory studies was also carried out :—

(20) Estimating the Thickness of a Floor at various points on the Profile of a Work.

(21) Correlation between Rainfall at Quetta and Water-Supply at Urak.

ANALYSIS OF PRESSURE PIPE OBSERVATIONS AT FEROZEPORE HEADWORKS

All the 29 bays of Ferozepore Weir are fitted with pressure pipes which are observed twice a month. The results are forwarded for examination to the Institute. The observations from 14th June 1940 to 15th June 1941 were analysed during the year.

At first the individual values of ϕ, the percentage residual head for each pipe were examined. These showed a lot of variation

during the period and so it (the period) was divided into sub-intervals (separately for each bay) in each of which the values appeared to be more homogeneous. The sub-intervals were so chosen that there were at least 2 observations in each group and that there were not more than six groups for any bay. The values in each group were averaged for each pipe separately and the values of r, the ratios of the average values of ϕ to the theoretical values were calculated.

These averages and the ratios r were studied and discussed separately for each bay in a note.

If r was constant in any group for all the pipes of a bay, it was concluded that the portion (1-r) of the total head H, was being cut off by the silt blanket upstream and the bay was recording pressures in perfect agreement with theory. If, however, the ratio increased from one pipe to the next, it meant that the gradient between the two pipes was flatter than it should be according to the theory. But if the ratio decreased, it meant that the gradient was steeper.

The discussion of the individual bays is not being reproduced, but only the main conclusions are given below :—

(*i*) Bays 25, 26 and 29 indicate the possibilities of cavities.

(*ii*) Bays 1, 19, 21, 27 and 28 show a flat gradient between pipes 7 and 8 and require watching. The other bays that show a flat gradient to a lesser extent are bays 2, 3 and 4.

(*iii*) Pipe No. 3 which is situated at the corner of the downstream face of the 1st line of sheet piles with the floor generally records higher (or equal) pressures than pipes 1 and 2 in bays 8, 12 and 13. This might be due to cracks in the bays extending up to pipe 3, unless it is choked. In the case of bay 8, the latest observations show a return to the normal condition.

(*iv*) Pipe No. 2 which is situated at the corner of the upstream face of the 1st line of sheet piles with the floor generally records higher values than pipe 1 in bay 17. Provided the pipe is not choked, cracks are indicated

(*v*) Pipe No. 2 in bays 1 and 2 and pipe 1 in bay 4 should be tested.

PRESSURE PIPE OBSERVATIONS FOR THE DEPRESSED BAYS OF ISLAM WEIR.

The pressure pipe observations for bays 11, 16 and 21 for the period April 1938 to December 1940 were analysed last year and are mentioned in the annual report of the Statistical Section for the year ending April 1941. As a result of the analysis, it was recommended that the pressure pipe data (for the pipes proposed to be put in) might be accumulated and be examined to verify the analysis given in the report.

This recommendation was accepted by the Chief Engineer and the Superintending Engineer, Nili Bar Circle, was instructed by him to supply the pressure pipe observations every month to the Research Institute. The data for May, June and September were received and examined during the year.

It was seen that all the bays where pipe No. 1 had been observed showed a greater drop of pressure than that expected on theoretical grounds. The reason for this might be that when the bays originally numbered 11 to 16 collapsed in 1929, the depressed bays were built on the debris which could not be taken out. The soil below the section of the bays between pipes 9 and 10 was likely to be much denser than that upstream. This would account for the greater drop of pressure between pipes 9 and 10. This, however, would not endanger the safety of the weir.

The fall expected, allowing for loss of head at entry and calculated on the assumption that the sub-soil was homogeneous, is compared below for each bay with the actual fall between pipes 9 and 10.

Bay No	1st May 1941 to 30th June 1941		4th September 1941 to 6th September 1941		11th September 1941 to 27th September 1941	
	Actual Fall	Expected Fall	Actual Fall	Expected Fall	Actual Fall	Expected Fall
11	18·8	6·3	19·7	7 8	18·0	6·6
14	14·2	8 3	16 2	10 4	17·2	9·0
16	14·9	7·5	12 5	8·8	14·6	7·4
18	16·8	7·8	14 5	8 8	16·1	7·7
20	13 5	8 0	12·6	9·2	14·6	8 0
21	17·9	7 8	17·7	9·4	17 8	8·1

N.B.—The expected fall has been calculated by multiplying the average values of the percentage residual head for pipes 1 to 9 by the difference in the theoretical values of ϕ between pipes 9 and 10.

DESIGN OF WORKS ON CLAY OR SAND-CLAY FOUNDATIONS

The design of works on clay or sand-clay foundations were discussed in a note prepared for the Eleventh Meeting of the Research Committee of the Central Board of Irrigation, July 1941. The summary of the conclusions is given below :—

(a) (i) For works on permeable homogeneous clay soils the design should be as if sand were substituted for clay. The uplift for a work formed by a floor and a toe pile can be found graphically for any point on the floor; for more complex profiles, the rules in Central Board of Irrigation Publication No. 12, may be followed.

(ii) The depth of toe pile, d, which for any profile, will give a gradient G, with a head H, and floor length L, is given by the formula—

$$d = \frac{H^2}{G\sqrt{10\,H^2 + 25\,G^2}}$$

(iii) The range for gradients for permeable clayey soils, formulated by Khosla —$\frac{1}{4}$ to 1/5—gives too wide a range for d. It would be advisable to link G more closely with the properties of the clayey soils.

(iv) As the permeable clay is liable to at least the same erosion as sand, the likely scour d/s of a work on such a soil may be such as to require a lower value of G in design than 1/5.

(b) (i) For works on impermeable clays, the need for a cut-off under the crest, recommended by Khosla, is not apparent, as with no flow in the clay, there is no danger of piping.

(ii) If the impermeable clay is liable to scour, the cut-offs at heel and toe must be provided to prevent slipping of the masonry into scour holes.

(c) If the work rests on a mixture of sand and clay, the design should be the same as for a work on fine sand.

(d) (i) When the work rests on a sand layer underlain by clay below which again there is sand, Khosla's rules should be followed : (1) Drive u/s pile into clay but not through it, (2) Do not drive d/s pile into clay .

(ii) In designing the d/s floor, remember that if the u/s pile leaks, the pressures will go up considerably.

(e) (i) For a pileless floor resting on sand of finite depth under lain by infinitely deep clay, the conditions are similar to those in D'Arcy and Clibborn's experiments, if the depth is small compared to floor length. The pressures along the floor tend to fall linearly except for rapid losses at entry and exit.

(ii) The Creep Theory while formulating the law of linear fall of pressure erred in omitting to see the relative smallness of sand depth in D'Arcy and Clibborn's experiments as compared with works resting on very deep sand.

(iii) For works on sand of finite depth, Khosla's rule may be followed. The u/s pile goes into clay, the d/s does not!

(f) If the work rests on a thick layer of clay, below which there is sand, the
alone. If the lay
sand; but allowanc
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GATED FLOW IN UNDERSLUICE MODELS

The correct estimation of discharge passing through a given undersluice bay—with a given opening and a given level of water in the river—is necessary, among other things, for a proper regulation of flow in the river, and of silt entry into the canal. Experiments were carried out in the Hydraulic Laboratory to find, by means of models, the dependence of the discharge through an undersluice, on the various hydraulic elements and ratios which are usually considered as characterising the flow under gated conditions.

Details of Experiments—The experiment was carried out in one of the flumes, the width of which was 3·5′. This was also the width of the orifice throughout the entire series of observations. The length of the flume upstream of the gate was sufficient to ensure uniform flow towards the gate line.

The height of the opening, O, was varied by fixing the bottom of the gate at different levels. With every given height, the discharge was so adjusted that the depth, H, of the water upstream of the gate was a multiple of the height of the opening.

The discharge, Q, through the orifice was estimated by passing the stream over a specially calibrated weir placed in the flume. The weir gauge was read and the point corresponding to the gauge reading on the discharge gauge calibration curve (experimentally determined) of the weir was taken as the value of the discharge.

The temperature of water was also observed. It was found to be sensibly constant throughout the entire series of experiments.

In all 52 observations were taken.

Formula connecting Q, H and O.—The values of Q were logarithmically correlated with O and H. It was seen that the correlation was very high. (·9988) and that it led to the formula—

$$Q = 13{\cdot}753 H^{\cdot 5116} \cdot O^{\cdot 9203} \qquad \text{.. (1)}$$

The values of Q calculated from this agreed to within 3-4 per cent with those actually observed.

The range of variations of the indices of H and O was calculated from the theory of probability. It was found that the range for the index of H included the value ·5000 but that for the index of O did not include 1·0000.

The Discharge Co-efficient—It was shown by dimensional analysis that C, the discharge co-efficient (as usually defined) was dependent on B/H (B being the width of orifice), O/H, R (Reynold's number) and F (Froude number). *For the given data* the degree of dependence of C on B/H could not be evaluated owing to the constancy of B and the consequent absorption of the effect of B/H in those of R and F.

The values of C were next correlated with V, H and O. The correlation was high. (·9967) and the resulting formula was :

$$C = V^{\cdot 8999}\ H^{\cdot 450}\ O^{-\cdot 8564}$$

It was deduced that :

$$C = R^{-\cdot 0010}\ F^{\cdot 4504}\ K^{-\cdot 8564}\ \text{constant} \quad (2)$$

where $K = O/H$.

The apparently low degree of dependence of the discharge coefficient on the Reynold's number was explained by showing that :—

$$C = \frac{(g\ F)^{\frac{1}{2}}}{(1+F/2)^{3/2} - (1+F/2-K)^{3/2}} \quad (3)$$

This relation is independent of any changes in other elements—such as roughness of bed, width of opening, Reynold's number, etc. for the simple reason that it is an algebraic identity.

MEAN VELOCITY AND MEAN SILT POINTS

Introductory—The point of mean velocity on a vertical of a channel, i.e., the point on a vertical where the velocity equals the mean of the velocities *on that vertical* has been previously determined and been found to lie near about ·6 of depth on the vertical. Similarly the point of mean silt intensity of a vertical, i.e., the point where the intensity of silt in suspension equals the mean of the silt intensities on that vertical has previously been determined for the central vertical for a number of channels and found to lie nearly at ·6D.

The object of the present investigation was to determine a point on the central vertical of the X-section of a channel where the suspended silt content was such that its product with the discharge would give the total floating silt charge of the channel. The suspended silt content which satisfies the above criterion may be termed as the 'MEAN SILT' and the point on the central vertical where the content equals the mean silt may be defined as the 'MEAN SILT POINT ON THE CENTRAL VERTICAL.'

As the calculation of the mean silt point involves the determination of the discharge or, what comes to the same thing, of the mean velocity of the channel and a knowledge of the velocity distribution on the central vertical, the 'MEAN VELOCITY POINT ON THE CENTRAL VERTICAL' (i.e. the point on the central vertical where the velocity equals *the mean velocity for the whole section*) was also determined.

Silt and Velocity Observations.—On five verticals of each of the following channels observations were made at every one-tenth of depth from ·1D to ·8D (or to ·7D, if the total depth at the point of sampling was less than 3·75′ as in this case the sampler touched the bottom of

the channel) for determination of silt in suspension under the directions of the Mathematical Officer. Simultaneous observations for determination of velocity were also made at ·1D, ·2D, ...·9D.

Name of Channel	R. D.	Number of Observations available
Lower Gugera Branch .	6,000	21
Burala Branch	6,000	21
Mian Ali Branch ..	95,000	28
Shah Kot Distributary ..	12,000	17
Khurrianwala Distributary ..	5,000	23

The calculations were done for 6 observations of each channel, i.e., 30 observations in all and the mean silt points were determined for medium silt only, i.e., the silt which has particles lying between ·075 mm. and ·2 mm.

The results showing mean velocity point, mean velocity, mean silt point and mean silt are given in Table LII. Table LIII gives the averages of the values in Table LII.

Conclusions—The results show that the mean velocity and mean silt (grade ·075 to ·2mm.) points on the central vertical differ from channel to channel and the standard deviation for observations on the same channel for mean velocity point (M. V. P.) is ·047D and for mean silt point (M. S. P.) is ·068D.

This means that if the silt sampling is to be done by taking one sample on the central vertical, we will have to determine the point separately for each channel and that the difference between the M. S. P. on any particular day and the average M. S. P. (if the average is taken from a large number of observations) will not exceed ·14D in 95 per cent of the cases. The same conclusion applies to mean velocity except that in this case the difference between M. V. P. for a particular observation and the average M. V. P. will not exceed ·09D in cent of the cases.

If for all the 5 channels, the sampling were to be on the central vertical (this is the average point for the the value of medium silt intensity obtained is likely to average silt intensity by more than 10 per cent in number of observations. In the case of velocity, if

The values of C were next correlated with V, H and O. The correlation was high. (·9967) and the resulting formula was :

$$C \doteq V^{\cdot 8999} \; H^{\cdot 1450} \; O^{-\cdot 8564}$$

It was deduced that :

$$C = R^{-\cdot 0010} \; F^{\cdot 4504} \; K^{-\cdot 8564} \text{ constant} \ldots \quad (2)$$

where $K = O/H$.

The apparently low degree of dependence of the discharge coefficient on the Reynold's number was explained by showing that :—

$$C = \frac{(g\ F)^{\frac{1}{2}}}{(1+F/2)^{3/2} - (1+F/2-K)^{3/2}} \quad \ldots \quad (3)$$

This relation is independent of any changes in other elements—such as roughness of bed, width of opening, Reynold's number, etc. for the simple reason that it is an algebraic identity.

MEAN VELOCITY AND MEAN SILT POINTS

Introductory—The point of mean velocity on a vertical of a channel, i.e., the point on a vertical where the velocity equals the mean of the velocities *on that vertical* has been previously determined and been found to lie near about ·6 of depth on the vertical. Similarly the point of mean silt intensity of a vertical, i.e., the point where the intensity of silt in suspension equals the mean of the silt intensities on that vertical has previously been determined for the central vertical for a number of channels and found to lie nearly at ·6D.

The object of the present investigation was to determine a point on the central vertical of the X-section of a channel where the suspended silt content was such that its product with the discharge would give the total floating silt charge of the channel. The suspended silt content which satisfies the above criterion may be termed as the 'MEAN SILT' and the point on the central vertical where the content equals the mean silt may be defined as the 'MEAN SILT POINT ON THE CENTRAL VERTICAL.'

As the calculation of the mean silt point involves the determination of the discharge or, what comes to the same thing, of the mean velocity of the channel and a knowledge of the velocity distribution on the central vertical, the 'MEAN VELOCITY POINT ON THE CENTRAL VERTICAL' (i.e. the point on the central vertical where the velocity equals *the mean velocity for the whole section*) was also determined.

Silt and Velocity Observations.—On five verticals of each of the following channels observations were made at every one-tenth of depth from ·1D to ·8D (or to ·7D, if the total depth at the point of sampling was less than 3·75′ as in this case the sampler touched the bottom of

the channel) for determination of silt in suspension under the directions of the Mathematical Officer. Simultaneous observations for determination of velocity were also made at ·1D, ·2D, ...·9D.

Name of Channel	R. D.	Number of Observations available
Lower Gugera Branch ..	6,000	21
Burala Branch .	6,000	21
Mian Ali Branch ..	95,000	23
Shah Kot Distributary ..	12,000	17
Khurrianwala Distributary ..	5,000	23

The calculations were done for 6 observations of each channel, i.e., 30 observations in all and the mean silt points were determined for medium silt only, i.e., the silt which has particles lying between ·075 mm. and ·2 mm.

The results showing mean velocity point, mean velocity, mean silt point and mean silt are given in Table LII. Table LIII gives the averages of the values in Table LII.

Conclusions—The results show that the mean velocity and mean silt (grade ·075 to ·2mm.) points on the central vertical differ from channel to channel and the standard deviation for observations on the same channel for mean velocity point (M. V. P.) is ·047D and for mean silt point (M. S. P.) is ·068D.

This means that if the silt sampling is to be done by taking one sample on the central vertical, we will have to determine the point separately for each channel and that the difference between the M. S. P. on any particular day and the average M. S. P. (if the average is taken from a large number of observations) will not exceed ·14D in 95 per cent of the cases. The same conclusion applies to mean velocity except that in this case the difference between M. V. P. for a particular observation and the average M. V. P. will not exceed ·09D in 95 per cent of the cases.

If for all the 5 channels, the sampling were to be done at ·5D on the central vertical *(this is the average point for the 5 channels)*, the value of medium silt intensity obtained is likely to differ from the average silt intensity by more than 10 per cent in nearly half the number of observations. In the case of velocity, if the observations

are taken at ·7D on the central vertical, the value obtained will differ from the mean velocity by more than 5 per cent in nearly a quarter of the number of observations.

SAMPLER Vs. SAMPLER (SUSPENDED SILT)

Details of Samplings—Observations were carried out at R. D. 1,600 of Upper Bari Doab Canal, Main Line for comparing the Puri, Binkley and Uppal samplers with the Bottle sampler. The last one is the standard sampler that is being used by the Institute. The description of the other samplers is given in the annual report of the Mathematical Section for the year ending April 1941.

The samples were taken at a point 31′ from the left bank. For each sample taken by the Bottle sampler 6 fillings were taken and mixed together. First of all a sample was taken by the Bottle sampler at ·1D, this was followed by a sample by the other sampler (say. Puri Sampler), then another by the Bottle sampler followed again by the Puri Sampler ; this combination was repeated once again. Such sets of samples were also taken at ·2D, ·3D, ·4D, ..., ..., ·8D making in all 48 samples per day. Observations for Puri versus Bottle sampler were taken for 6 days, for Binkley versus Bottle sampler for 5 days and for Uppal versus Bottle sampler for 3 days.

Analysis of Data.—The data were analysed by the method of the 'Analysis of Variance'. This showed that the variance due to all the primary factors, namely, sampler, dates and depths and their interactions were all significant on 1 per cent level. It meant that—

(*i*) The silt intensities obtained by the bottle sampler and the other sampler were different.

(*ii*) The differences between the two samplers (bottle sampler and the other sampler) were different at different depths.

(*iii*) The differences between the two samplers at a particular depth were different on different dates.

(*iv*) The variation in the difference between the two samplers at different depths was different on different dates.

The average values of the silt content obtained by the two samplers showed that at every depth, the bottle sampler gave a higher silt content than the Puri and Uppal samplers and a lower silt content (except at ·1D) than the Binkley sampler.

The next point to be considered was that of the relative constancy of the two samplers. For this purpose, the residual sum of squares was split into two parts—the portion contributed by the bottle sampler and the portion contributed by the other sampler. From this the standard errors within the samplers were calculated and compared statistically.

It was found that :—

(i) The standard error in the case of Puri sampler was not significantly different from that of the bottle sampler.

(ii) The standard error in the case of the Uppal sampler was more than that of the bottle sampler. But if the set of observations for 14th September 1941 be omitted, the two were not significantly different.

(iii) The standard error in the case of the Binkley sampler was more than that of the bottle sampler.

Below are given, the mean silt content (m), the standard error (σ), and the coefficient of variability (c. v. $= \frac{\sigma}{m} \times 100$) for the samplers (average for all depths and dates) :—

—	m		σ		c. v.	
	Other Sampler	Bottle Sampler	Other Sampler	Bottle Sampler	Other Sampler	Bottle Sampler
1 Puri *versus* Bottle Sampler.	097	121	0158	0138	16·3	11·4
2 Binkley *versus* Bottle Sampler	·177	·135	0360	0177	20·3	13·1
3. Uppal *versus* Bottle Sampler	·097*	·124*	0118*	0114*	12·2*	9·2

*Excluding observations at 8D for 14th September 1941

Conclusions.—The Puri and the Uppal samplers gave a lower silt content than the bottle sampler. The Binkley sampler gave higher values than the bottle sampler as far as these observations were concerned.

The results given by the Binkley sampler were more variable than those given by the bottle sampler. While in the case of the Puri and Uppal samplers, the variability was of the same order of magnitude as for the bottle sampler.

ROLLING SILT OBSERVATIONS AT TIBRI

A sampler for measuring the quantity of rolling silt has been devised by Dr. Bose. A description of this is given in the Annual Report of the Mathematical Section for the year ending April 1940. Some observations were carried out this year by the Executive Engineer, Gurdaspur, with the sampler. It was found that on all the days except one, the quantity of rolling silt had become constant before 60 minutes. A curve of the type $V=a(1-be^{-kt})$ was fitted to the data for the day when the rolling silt had not become constant. It was found that the time for which the volume would be 5 per cent of the limiting value was 2 hours nearly. If similar observations were to be continued, it was suggested that the maximum period of observation should be increased to 120 minutes instead of 60 minutes.

are taken at ·7D on the central vertical, the value obtained will differ from the mean velocity by more than 5 per cent in nearly a quarter of the number of observations.

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contention was that the Kutter equation is unnecessarily complicated, and that it appears to have no advantage over simpler equations. He thought that 'if it is desired to replace the Kutter equation as closely as possible by an exponential equation, it is firmly established beyond all doubt that the Manning equation is the best substitute to employ. If, on the other hand, it is contended that both the Kutter and Manning equations require some slight adjustment by canal engineers when assessing the value of N on large canals, then that adjustment is best made by slightly modifying the power of R, and employing the Lea-Lacey equation instead'.

After some discussion it was resolved by the meeting of the Research Committee that 'The Research Departments of different provinces should examine the existing data from canals and rivers with a view to determining the most probable or other relationship'.

At the Tenth Annual Meeting, the following suggestion was approved by the Research Committee.

"From simultaneous values of V, R and S, the values of Kutter's, Lacey's, Manning's and Barnes-Forchheimer's N's should be calculated and the following studies carried out :—

(*a*) How far do these indices agree between themselves ?

(*b*) Do the changes in these indices from channel to channel indicate a real difference in condition ?

(*c*) How do the actual values for rugosity compare with those adopted for original design ? This will involve a study of the history of each channel in regard to remodelling, silt extraction from the parent or the channel, etc."

The Formulae Proposed for Investigation.—Using initial letters of their author's names as suffixes to distinguish the various rugosity coefficients, the formulae in foot-second units are :—

$$N_B = (1 \cdot 4282/V) \;.\; R^{\cdot 70} \; S^{\frac{1}{2}},$$

$$N_K = 3 \cdot 622/\sqrt{Z^2 + 7 \cdot 244\ Y} + Z$$

where $Z = V/(RS)^{\frac{1}{2}} — 41 \cdot 6 — \cdot 00281/S$

and $Y = (V/RS^{\frac{1}{2}}) \cdot (41 \cdot 6 + \dfrac{\cdot 00281}{S})$

$$N_L = \frac{1 \cdot 3458}{V} \;.\; R^{\frac{3}{4}} S^{\frac{1}{2}},$$

and $$N_M = \frac{1 \cdot 4858}{V} \;.\; R^{\frac{2}{3}} S^{\frac{1}{2}}$$

LACEY'S BASIC REGIME THEORY.

In a note on 'Basic Regime Theory' Mr. Lacey has tried to explain "a number of anomalies which arise in model work and a number of divergencies" by the introduction of a new variable $E(=P/W)$ in his basic equations.

His previous basic formulae not involving b_i were :—

$$P = K\sqrt{Q} \quad \ldots \quad (1)$$

$$\text{and } V = 16{\cdot}05\,(R^2 S)^{\frac{1}{3}} \quad \ldots \quad (2)$$

These have been changed to—

$$P = K E\sqrt{Q}$$

$$\text{or } W = K\sqrt{Q} \quad \ldots \quad (1')$$

$$\text{and } V = 16{\cdot}05\,(ER^2 S)^{\frac{1}{3}} \quad \ldots \quad (2')$$

These formulae were tested on 17 Punjab channels to see if (1′) and (2′) fitted the data better than (1) and (2). In Table LIV are given the values of $P/\sqrt{Q}$, $W/\sqrt{Q}$, and their percentage deviations from their mean values. In Tables LV are given the values of $V/(R^2\ S)^{\frac{1}{3}}$, $V/(E\,R^2 S)^{\frac{1}{3}}$ and their percentage deviations from the respective means.

Table LIV shows that the maximum deviation of $P/\sqrt{Q}$ from its mean is 19·1 per cent and that of $W/\sqrt{Q}$ is 18·2 per cent. But the coefficient of variation, which is a measure of the root-mean-square deviation from its mean expressed as a percentage of the mean value, is slightly lower than that of $W/\sqrt{Q}$,—the actual values being 7·4 per cent and 8·5 per cent, respectively.

The maximum deviation of $V/(R^2\ S)^{\frac{1}{3}}$ is 18·2 per cent and that of $V/(ER^2\ S)^{\frac{1}{3}}$ is 17·4 per cent. The coefficients of variations are nearly equal, being 11·5 per cent and 10·8 per cent, respectively.

As the variation in the value of E for these channels is rather small (E varies from 1·03 to 1·18), we could not expect the old and the new equations to give wider differences. The data show that for the 17 channels the average percentage variation of $P/\sqrt{Q}$ (as measured by the coefficient of variation) is somewhat less than that of $W/\sqrt{Q}$ though the difference is not significant. It is now proposed to find by the method of correlation on a larger set of data if the introduction of E improves the correlation.

THE BEST FLOW FORMULA.

Introductory.—A note on the 'Replacement of the Kutter equation by a simpler exponential formula,' was presented by Mr. Lacey at the Ninth Annual Meeting of the Research Committee of the Central Board of Irrigation held at Simla in July, 1939. Mr. Lacey's

FIG. 2.10

COMPARISON OF THE VALUES OF THE KUTTER'S N
FOR SOME OF THE REGULARLY OBSERVED
CANAL SITES IN THE PUNJAB

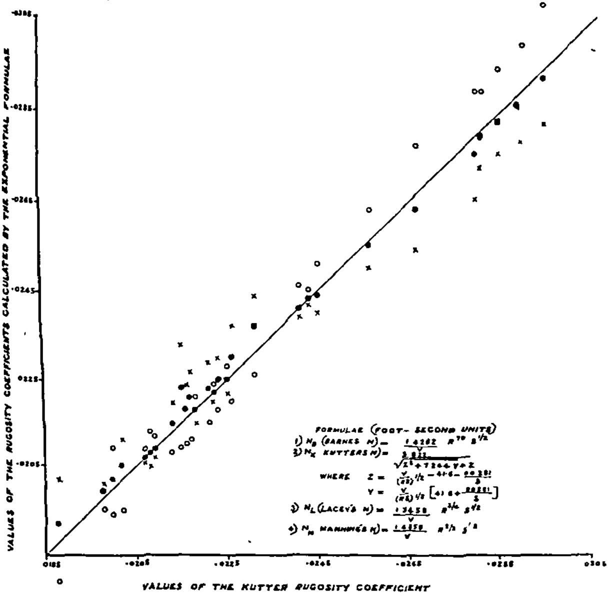
WITH

THE VALUES OF BARNES', LACEY, AND MANNING'S N'S

NOTATION -

BARNES' N = •
LACEY'S N = o
MANNING'S N = x

N.B. THE STRAIGHT LINE IS DRAWN AT A 1:1 SLOPE
THE POINT IN WHICH THE JOIN OF •, o, AND x
CUTS IT GIVES THE VALUE OF KUTTER'S N FOR THE SITE



Comparison of N_B, N_K, N_L and N_M for 42 regularly Observed Punjab Sites.—This year the analysis was confined to part (*a*) of the resolution passed at the Tenth Annual Meeting of the Research Committee of the Central Board of Irrigation, viz., "How far do the values of N_B, N_K, N_L and N_M, calculated from simultaneous values of V, R and S agree between themselves?" The data refer to 42 sites, regularly observed for a fairly long period and believed to be stable. The average values of V, R and S for these sites are given in Columns 5—7 and of N_B, N_K, N_L and N_M in Columns 8—11 of Table LVI. For some of the sites the values of N_B, and N_M are plotted against those of N_K (Fig. 210).

The values of N_B, N_K, N_L and N_M were subjected to a comparative study in the following manner:—

(*a*) *Mean Values for all the 42 Sites.*—The mean values for the indices were calculated and it was found that for all practical purposes, these could be taken to be mutually equal and to be not very different from ·0225.

(*b*) *Standard Deviations of Differences for Individual sites.*—The values of the standard deviations (S. D) of the differences for various pairs of indices were all small, but the smallest by far was that for the S. D. of N_B—N_K.

(*c*) *Correlations with other Indices.*—The correlation coefficients between the possible pairs of indices were calculated. Though all the correlations were high, the highest was that between N_B and N_K.

It was also noticed that if the set of the 3 correlations between each index and the rest was considered as a whole, the best performance was put up by N_B.

(*d*) *Formulae for estimating each Index from any of the other Three.*—It was noticed that the regression coefficients of N_L on the other three indices were all greater than unity. It means that any change in N_B, N_K, or N_M leads to a proportionately larger change in N_L, which should therefore be regarded as more variable than any of the other three. The opposite was the case with N_M.

This is also brought out if we consider the ranges of N_B, N_K, NL, and N_M from Table LVI.

(c) *Examination of Tabulated Values.*—From Table LVI it is apparent that N_K lies almost exactly midway between N_L and N_M. This is due to the fact that—

(i) For these 42 sites N_K is very close to N_B, and

(ii) N_B must always lie between N_L and N_M, as the index of R in Barnes' Formula (viz. ·70) lies between the indices of R in the Lacey and Manning Formulae (viz. $\frac{3}{4}$ and $\frac{2}{3}$, respectively).

To sum up, it appears that, for the 42 regularly observed Punjab sites the various rugosity coefficients are generally of the same order of magnitude and not very different from each other, though the closest agreement is that provided by the Barnes and Kutter coefficients which are almost identical for most of the sites.

The Error Arising From Interchange of Indices.—As N_B and N_K lie between N_L and N_M, the greatest percentage difference can arise through calculating the velocity from the Lacey Formula with a rugosity index derived from the Manning Formula and vice versa.

The % differences are shown in column 12. It will be noticed that except for two small channels discharging less than 10 cusecs, the maximum difference does not exceed 10%.

DESIGN OF REGIME CHANNELS

The following correlations were carried out between the logarithms of various hydraulic elements for the average values of 42 regularly observed Punjab channels :—

Serial No.	Variables Correlated	Correlation Coefficient	Resulting Formula
1	Log R, log P, log S.	·992	$R = \cdot 18\, P^{\cdot 58} / (S \times 10^3)^{\cdot 35}$
2	Log R, log P, log Q.	·998	$R = \cdot 84\, Q^{\cdot 69} / P^{\cdot 68}$
3	Log R, log P, log m.	·987	$R = \cdot 46\, P^{\cdot 72} / (m \times 10^2)^{\cdot 26}$
4	Log D, log Q.	·985	$D = \cdot 65\, Q^{\cdot 31}$
5	Log D, log Q, log S.	·989	$D = \cdot 515 Q^{\cdot 27} / (S \times 10^3)^{\cdot 27}$
6	Log D, log Q, log m.	·988	$D = 1 \cdot 72 Q^{\cdot 34} / (m \times 10^2)^{\cdot 36}$
7	Log A, log Q.	·999	$A = 1 \cdot 145\, Q^{\cdot 85}$
8	Log A, log Q, log S.	·999	$A = 1 \cdot 12\, Q^{\cdot 85} / (S \times 10^3)^{\cdot 02}$

RUGOSITY COEFFICIENTS ON LINED CHANNELS

Some data on the Haveli Main Line were analysed last year and mentioned in the Annual Report of the Statistical Section for the year ending April 1941. It was suggested that daily observations be taken on the Haveli Main Line for a few months from which the relation between V, R and S should be deduced. The Chief Engineer ordered discharge observations on Haveli and Bikaner Canals to determine the value of the rugosity coefficient on lined channels. Data at the following sites were obtained :—

Canal	R. D.	Period
Haveli Main Line ..	80,000	6-12-41 to 31-12-41
Ditto	133,000	19-9-40 to 29-11-40
Ditto	142,000	1-1-41 to 31-7-41
Ditto	165,000	23-9-40 to 31-12-40
Ditto	220,000	25-4-41 to 31-7-41
Bikaner Canal	43,000	24-9-40 to 31-8-41
Ditto	188,000/ 188,764	1-10-40 to 10-1-41
Ditto	305,000	22-12-40 to 31-8-41
Ditto	343,000	22-9-40 to 21-12-40

For one of the sites—Bikaner Canal R. D. 188,000—the correlation coefficient between rugosity coefficient and discharge was worked out and it was found that there was no correlation. The average values of the rugosity coefficients—Kutter's (N_K), Manning's (N_M) and Lacey's (N_L)—together with their standard deviations were next calculated. These are given in Table LVII. As the standard deviations of the rugosity coefficients in some cases are appreciable, the average values of the coefficients should not be accepted for purposes

Three other correlations between (*i*) log A, log D, log S, (*ii*) log D, log Q, and (*iii*) log D, log Q, log S, were also worked out for data selected *at random* from full supply days for 47 Punjab Channels.

Two of the Formulae given above, namely—

$$A = 1.145\ Q^{\cdot 85}$$

$$\text{and } D = \cdot 515\ (Q/S \times 10^3)^{\cdot 27}$$

were selected by Dr. N. K. Bose and Mr. K. R. Erry for their Punjab Engineering Congress paper, 'Design of Channels in Alluvium' to replace the modified P—Q and R—Q relationships of Lacey.

DIFFERENCE BETWEEN 'SHOCK' AND 'DEVIATION FROM REGIME', FOR A CHANNEL IN ALLUVIUM.

In a note written for the Eleventh Research Meeting of the Central Board of Irrigation an attempt was made to clarify the idea of 'regime' and to distinguish between 'shock' and 'change in regime'.

In view of the need for, and the absence of an authoritative definition of, 'regime' the following definition was suggested for consideration.

"Regime is a state of equilibrium between the forces operative in a reach of a channel, flowing in and transporting its own silt. If these forces are generated solely by such flow and transport, the channel is in regime, free from 'Shock'; 'Shock' being defined as the operation of forces over and above those generated by the interaction of silt and water."

Lacey Criterion for 'Regime free of Shock'—For a channel in shock-free regime the Lacey ratio $(C_L) = V/16R^{\frac{2}{3}}S^{\frac{1}{3}}$, where V, R and S are mean velocity, hydraulic mean depth and water surface slope respectively, equals unity. But it has been found that this equality of C_L to unity is also realised on some unstable channels in the Punjab. Mr. Lacey has stated that this was very likely due to a balancing of shock present on unstable channels against a departure from regime. This leads to the important deduction that 'shock' and 'departure from regime' can exist simultaneously on the same channel. The separation of the effects due to these requires further investigation.

'Shock' and 'regime' should be studied more thoroughly, particularly through observations on channels where 'shock' and deviation from 'regime' are appreciable, i.e., on channels in bad order, and on unstable channels. This would show if the formulae developed from stable channels and channels in good order are real criteria of stability or good order, when applied to design of channels in alluvium.

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"Regime is a state of equilibrium between the forces operative in a reach of a channel, flowing in and transporting its own silt. If these forces are generated solely by such flow and transport, the channel is in regime, free from 'Shock'; 'Shock' being defined as the operation of forces over and above those generated by the interaction of silt and water."

Lacey Criterion for 'Regime free of Shock'—For a channel in shock-free regime the Lacey ratio $(C_L) = V/16R^{\frac{2}{3}} S^{\frac{1}{3}}$, where V, R and S are mean velocity, hydraulic mean depth and water surface slope respectively, equals unity. But it has been found that this equality of C_L to unity is also realised on some unstable channels in the Punjab. Mr. Lacey has stated that this was very likely due to a balancing of shock present on unstable channels against a departure from regime. This leads to the important deduction that 'shock' and 'departure from regime' can exist simultaneously on the same channel. The separation of the effects due to these requires further investigation.

'Shock' and 'regime' should be studied more thoroughly, particularly through observations on channels where 'shock' and deviation from 'regime' are appreciable, i.e., on channels in bad order, and on unstable channels. This would show if the formulae developed from stable channels and channels in good order are real criteria of stability or good order, when applied to design of channels in alluvium.

RUGOSITY COEFFICIENTS ON LINED CHANNELS

Some data on the Haveli Main Line were analysed last year and mentioned in the Annual Report of the Statistical Section for the year ending April 1941. It was suggested that daily observations be taken on the Haveli Main Line for a few months from which the relation between V, R and S should be deduced. The Chief Engineer ordered discharge observations on Haveli and Bikaner Canals to determine the value of the rugosity coefficient on lined channels. Data at the following sites were obtained :—

Canal	R. D.	Period
Haveli Main Line ..	80,000	6-12-41 to 31-12-41
Ditto	133,000	19-9-40 to 29-11-40
Ditto	142,000	1-4-41 to 31-7-41
Ditto	165,000	23 9 40 to 31-12-40
Ditto	220,000	25-4-41 to 31-7-41
Bikaner Canal	43,000	24-9-40 to 31-8-41
Ditto	188,000/ 188,764	1-10-40 to 10-1-41
Ditto	305,000	22-12-40 to 31-8-41
Ditto	343,000	22-9-40 to 21-12-40

For one of the sites—Bikaner Canal R. D. 188,000—the correlation coefficient between rugosity coefficient and discharge was worked out and it was found that there was no correlation. The average values of the rugosity coefficients—Kutter's (N_K), Manning's (N_M) and Lacey's (N_L)—together with their standard deviations were next calculated. These are given in Table LVII. As the standard deviations of the rugosity coefficients in some cases are appreciable, the average values of the coefficients should not be accepted for purposes

Three other correlations between (*i*) log A, log D, log S, (*ii*) log D, log Q, and (*iii*) log D, log Q, log S, were also worked out for data selected *at random* from full supply days for 47 Punjab Channels.

Two of the Formulae given above, namely—

$$A = 1.145\ Q^{\cdot 85}$$

$$\text{and } D = \cdot 515\ (Q/S \times 10^{3})^{\cdot 27}.$$

were selected by Dr. N. K. Bose and Mr. K. R. Erry for their Punjab Engineering Congress paper, 'Design of Channels in Alluvium' to replace the modified P—Q and R—Q relationships of Lacey.

DIFFERENCE BETWEEN 'SHOCK' AND 'DEVIATION FROM REGIME', FOR A CHANNEL IN ALLUVIUM.

In a note written for the Eleventh Research Meeting of the Central Board of Irrigation an attempt was made to clarify the idea of 'regime' and to distinguish between 'shock' and 'change in regime'.

In view of the need for, and the absence of an authoritative definition of, 'regime' the following definition was suggested for consideration.

"Regime is a state of equilibrium between the forces operative in a reach of a channel, flowing in and transporting its own silt. If these forces are generated solely by such flow and transport, the channel is in regime, free from 'Shock'; 'Shock' being defined as the operation of forces over and above those generated by the interaction of silt and water."

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'Shock' and 'regime' should be studied more thoroughly, particularly through observations on channels where 'shock' and deviation from 'regime' are appreciable, i.e., on channels in bad order, and on unstable channels. This would show if the formulae developed from stable channels and channels in good order are real criteria of stability or good order, when applied to design of channels in alluvium.

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of design but the maximum values which are likely to be realised on full supply days should be taken. These are given below :—

Canal	R. D.	Date	Q	N_K	N_M	N_L
Haveli Main Line	80,000	Supplies were very low throughout the period under observation				
Ditto ..	133,000	7-10-40	3,673	·0139	·0140	·015
Ditto ..	142,000	11-5-41	4,422	·0161	·0163	·017
Ditto ..	165,000	2-10-40 to 6-10-40	3,531	·0151	·0153	·016
Ditto ..	220,000	16-7-41	4,286	·0176	·0178	·019
Bikaner Canal ..	43,000	20 6-41	2,339	·0169	·0170	·018
Ditto ..	188,000	17-10-40 to 19-10-40	1,659	·0155	·0158	·016
Ditto ..	305,000	17-8-41	2,568	·0146	·0148	·015
Ditto ..	343,000	22/23-9-40	2,062	·0142	·0143	·015

Regarding Haveli Main Line R. D. 220,000 according to Executive Engineer, Main Line Division, "the main thing about this reach (R. D. 168,000 to 227,800) is that it gets affected by the gauge that is maintained above Sidhnai Dam for regulation purposes". It was found that there was no correlation between V and R at this site.

It appears, therefore, that the following values may be accepted for design of channels with similar types of lining to those examined.

$$N_L = \cdot 0180 \; - \; \cdot 0185$$

$$N_M \text{ or } N_K = \cdot 0165 \; - \; \cdot 0170$$

SILT ENTRY INTO CANALS AS AFFECTED BY THE SUPPLY AND THE REGULATION AT HEADWORKS.

Observations at Ferozepore headworks were made from June to September 1941 and different types of undersluice settings were tried. The data were examined in the light of the rules framed in 1939. It appeared that one of the rules required slight modification.

The bed silts taken from the canal showed that the silting in the head reach could not be ascribed to the coarseness of material entering the canal. It may possibly be due to the poor working head of the canal.

SILT SURVEYS

(i) The silt survey of the L. C. C. was started in 1938 in order to follow the changes that were taking place in the canals

of the Lower Chenab Canal system due to the construction of silt excluder tunnels at Khanki Headworks. The mean diameters of bed silts taken from various channels of the system were calculated and the values of slopes worked out from the Bose slope formula. These were forwarded to the Mathematical Officer for analysis.

(ii) Heavy silt having settled down in the head reach of the Nurpur distributary, silt excluding devices were fitted in 1932-33. The result was that the silt deposited in the head reach started moving downstream. A silt survey of the channel was carried out in Kharif 1941. The values of A, D and S (Area, Depth and Slope) were calculated from the Institute formulae and compared with the actuals. The results were put up to the Mathematical Officer.

RELATION BETWEEN CLAY % OF A SOIL AND ITS CAPACITY FOR ABSORBING WATER

Following a study of observed and plotted values of C (clay %) and V (volume of soil absorbing 1 c.c. of water—for the method of determination of V, please see page 221 of the Annual Report for the year ending April 1941) for 101 samples, the following relation was derived by correlation.

$$C = \frac{17 \cdot 144}{V - 3 \cdot 245} + 2 \cdot 149 \quad \ldots \quad \ldots \quad (1)$$

This formula gave values of C markedly different from the actual ones, where V had some of the smallest values. The following alternative empirical formula was found to be more satisfactory in this respect.

$$C = \frac{30}{V - 2 \cdot 7} - \cdot 5 \quad \ldots \quad \ldots \quad (2)$$

Formula (2) gave results in good agreement with actuals if V was higher than $5 \cdot 0$; for V below $4 \cdot 0$, it gave the order of magnitude correctly, though the individual values might differ appreciably.

It was found that the following rules were satisfied by a large number of values.

(i) If V exceeds 7, C is less than 7.

(ii) If V exceeds 5, but not 7, C is less than 13, but bigger than 7.

(iii) If V exceeds 4, but not 5, C is less than 25, but bigger than 13.

It was concluded, therefore, that when the order of the magnitude alone (of C) is to be found, the above rules could be applied, provided V was in excess of $4 \cdot 0$.

If V was less than $4 \cdot 0$, C was, in a large majority of cases higher than 25. It would be preferable, however, in such cases to find C directly.

EFFECT OF SOIL FACTORS ON YIELD OF WHEAT.

The yield of wheat was correlated with various soil characteristics—contents of manganese, phosphates, total soluble salts, exchangeable sodium, potassium, calcium, and magnesium, and for their pH values—for 6 districts of the Punjab. It was found that a negative significant correlation (= —·58) existed between the manganese content and the yield. An equally positive correlation was found between the phosphate content of soils and the yield. The correlations of yield with other characteristics are not significant.

CHANGES IN THUR AREA IN 8 DISTRICTS OF THE PUNJAB.

The rate of increase of thur in eight districts of the Punjab during the years 1931-32 to 1939-40 has been calculated previously and mentioned in the last annual report. The rate of increase was calculated on the figures of total thur within the boundaries of each district irrespective of whether the area was commanded or not commanded by the canals irrigating the district.

Now the rate of increase was worked out on the figures of the thur *within the culturable commanded area* of the canals. It was found that the average rate of increase for all the districts taken together, was nearly 17,800 acres per year. This was ·19 per cent or about 1/500 of the C. C. A. This rate of increase was about ⅔rds of the rate of increase within the entire district area, viz. 28,200 acres, which was ·30 per cent or about 1/330 of the C. C. A. This ratio of ⅔ was also approximately satisfied by Sheikhupura, Gujranwala, Shahpur, Montgomery and Lyallpur districts taken individually.

Next the rate of increase within the culturable commanded area *by canals* was calculated. The rate of increase was the largest in the Lower Chenab Canal area being 8,950 acres or ·31 per cent of the culturable commanded area. This was followed by Upper Chenab Canal area where the rate of increase was 7,080 acres or ·50 per cent of the C. C. A.

FACTORS AFFECTING AREA IRRIGATED

Two investigations were carried out under this head :—

(*a*) The effect of increase in supply on the area irrigated by 6 channels in the Khanewal Division.

(*b*) The analysis of factors affecting area irrigated on the Lower Chenab Canal.

(*a*) *The effect of increase in supply on the area irrigated by 6 channels in the Khanewal Division.*—Following the hailstorm in April, 1937, it was decided, t[illegible]
Kharif in the channels of the L[illegible]
stricken area because of the [illegible]
the actual results achieved, [illegible]

Bari Doab Canal, remarked that the area irrigated was controlled by factors other than a 10 per cent variation in supply (e.g. rainfall and prices) so long as the supply did not fall below a certain limit.

The case was referred by the Chief Engineer to the Institute for investigating statistically as to what results could be expected from increases in supplies.

From an examination of the data for three channels out of the affected area and three out of the unaffected area in the Khanewal Division for the years 1933 to 1937, it was concluded that :—

(i) The area irrigated by the 6 channels was increasing.

(ii) So long as the supply did not fall below a certain minimum, the area irrigated was independent of the discharge utilized.

(iii) There did not appear to be any obvious relation between the area *irrigated during Kharif and rainfall during* the same period.

(iv) The ratio of the area irrigated under a particular crop to the total area irrigated was probably affected by the change in the price of the crop.

The conclusions drawn above were, however, only tentative. It was proposed to investigate the effect of factors such as prices and discharge utilized on the area irrigated by the Lower Chenab Canal, the data for which were available for a fairly long period.

(b) *The Analysis of Factors affecting Area Irrigated on the Lower Chenab Canal—Selection of Variables.*—In order to find the possible factors that affect the area irrigated, the administration reports of the Lower Chenab Canal for the years 1901-02 to 1938-39 were studied and the factors selected for correlation with (i) area irrigated during Kharif, and (ii) area irrigated during rabi were :—

(i) *Area irrigated during Kharif*—

Rainfall—divided into three periods April-May, June-July and August-September.

Discharge utilised during Kharif.

Price of Cotton.

(ii) *Area irrigated during Rabi*—

Rainfall—divided into three periods October-November December-January, and February-March.

Discharge utilized during rabi.

Price of wheat.

Price of rapeseed.

The values of area irrigated, discharge utilized and rainfall were taken from the printed volumes of the "Irrigation Statistics." The prices at Lyallpur at harvest time were obtained from the Director of Land Records.

As the rainfall figures were available only since the year 1908-09, the period from 1908-09 to 1938-39 was selected for analysis.

Method of Analysis.—When dealing with figures arranged in order at equal intervals of time, e.g., annual figures, the outstanding difficulty in the direct application of the correlation coefficient is that the variables considered generally exhibit progressive changes during the period available. Such changes in this particular case may be due to some such factors as :—

(i) Development of irrigation and improved distribution of water and other facilities.

(ii) Removal of harmful effects of irrigation by drainages etc.

(iii) Economic factors which affects the prices in general and not of any particular commodity in particular.

ay be obtained by smoothing out
data and drawing smoothing lines
by the eye. But the method most commonly used in statistics is that of fitting polynomials, by the method of least squares, of the 1st, 2nd, 3rd or higher order and retaining that by which the progressive change is best represented.

Polynomials up to the fifth order were fitted to all the variables and as in the case of the area irrigated the coefficients of the fifth order were significant, by statistical tests, it was decided to retain the first five terms to express the progressive change in each case. Terms of higher order were not considered necessary.

The rainfall did not show any progressive change during the period. In the case of other variables the regression curves of the progressive change are plotted in Figs. 211 to 217.

The deviations of the actual values from the progressive change of areas irrigated during kharif and rabi were correlated with similar deviations of the variables given above. The correlation coefficients are given in Tables LVIII and LIX.

Significance of the Progressive Changes.

(i)
irrigated
of the are
of the variation. A similar trend is noticed in the case of the discharge utilized during kharif.

It appears that the factor mostly responsible for this progressive increase is the development of irrigation and improved distribution of water.

(ii) *Area irrigated during rabi*—It may be noted that the trend from the year 1923 onwards is similar to the trend in the case of prices of cotton, wheat and rapeseed.

The correlations of the *actual* values of the area irrigated with the prices at previous harvest of cotton, wheat and rapeseed are ·42,

FIG. 2.11

LOWER CHENAB CANAL

CORRELATION OF AREA IRRIGATED DURING KHARIF WITH PRICE OF COTTON AT PREVIOUS HARVEST AND DISCHARGE UTILIZED

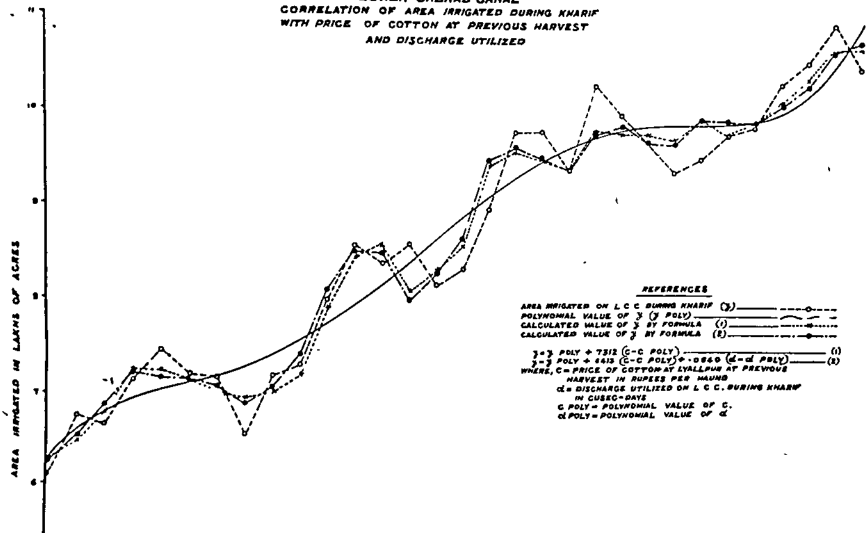


FIG. 212

LOWER CHENAB CANAL

DISCHARGE UTILIZED DURING KHARIF

(ACTUAL AND POLYNOMIAL)

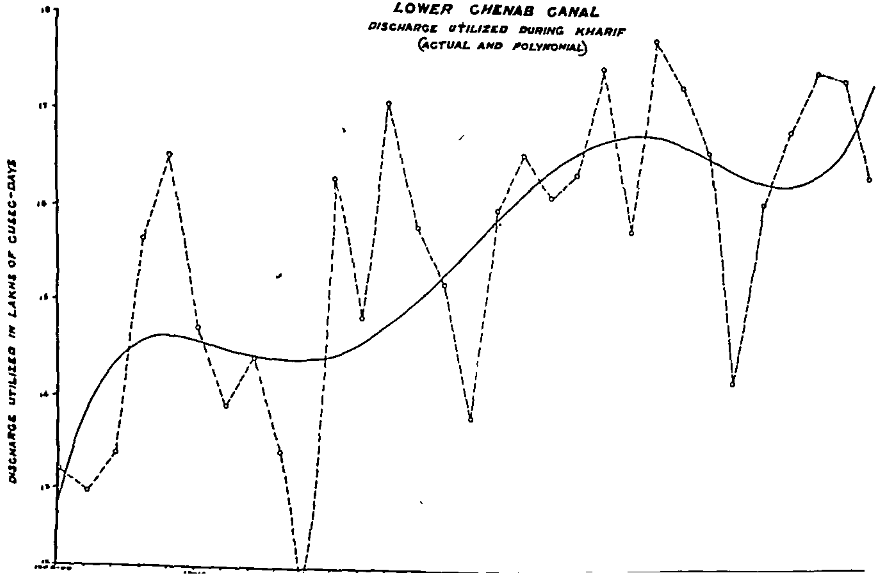


FIG. 2.13

PRICE OF COTTON AT LYALLPUR AT HARVEST TIME
IN THE YEAR ENDING 15 TH JUNE
(ACTUAL AND POLYNOMIAL VALUE)

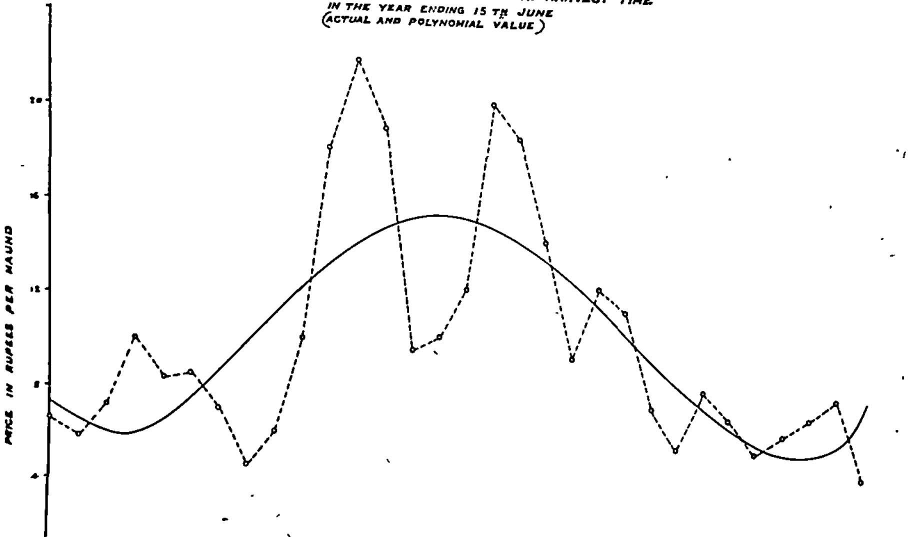


FIG. 214

LOWER CHENAB CANAL

CORRELATION BETWEEN AREA IRRIGATED DURING RABI AND DISCHARGE UTILIZED DURING RABI

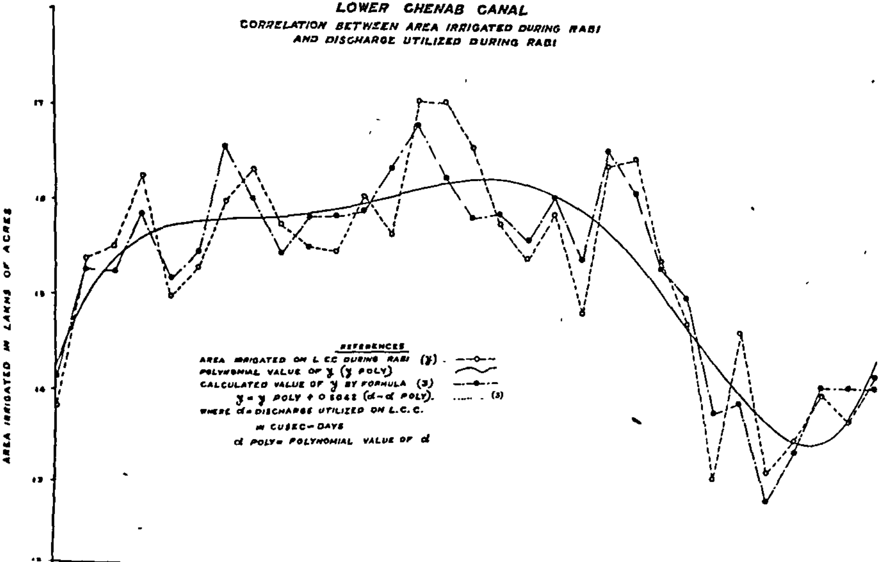


FIG. 2.15

LOWER CHENAB CANAL

DISCHARGE UTILIZED DURING RABI

(ACTUAL AND POLYNOMIAL)

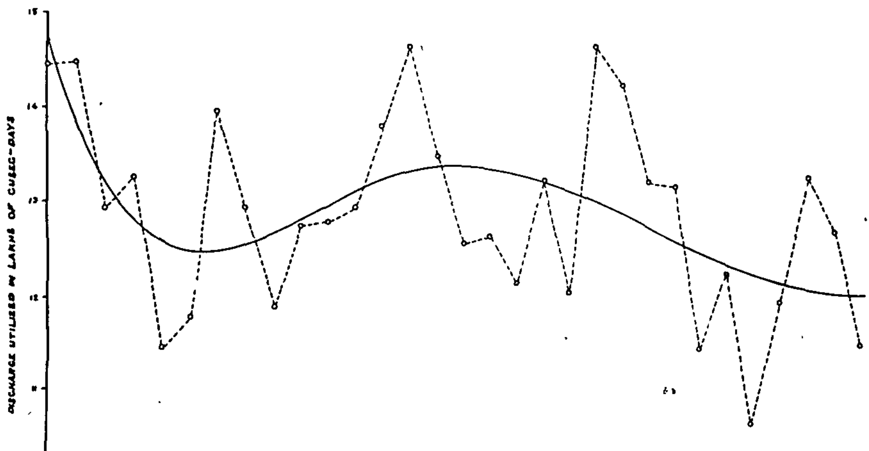


FIG. 216

PRICE OF WHEAT AT HARVEST TIME
IN THE YEAR ENDING 15TH JUNE
(ACTUAL AND POLYNOMIAL VALUE)

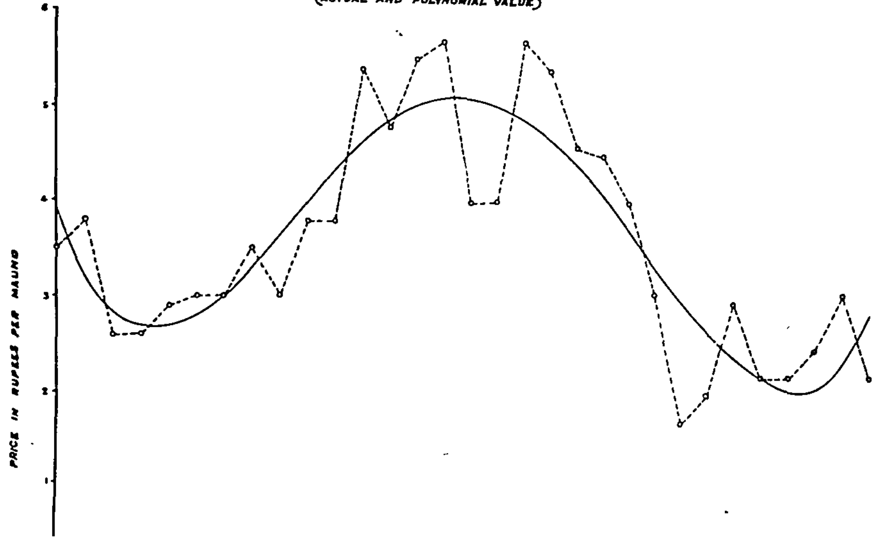
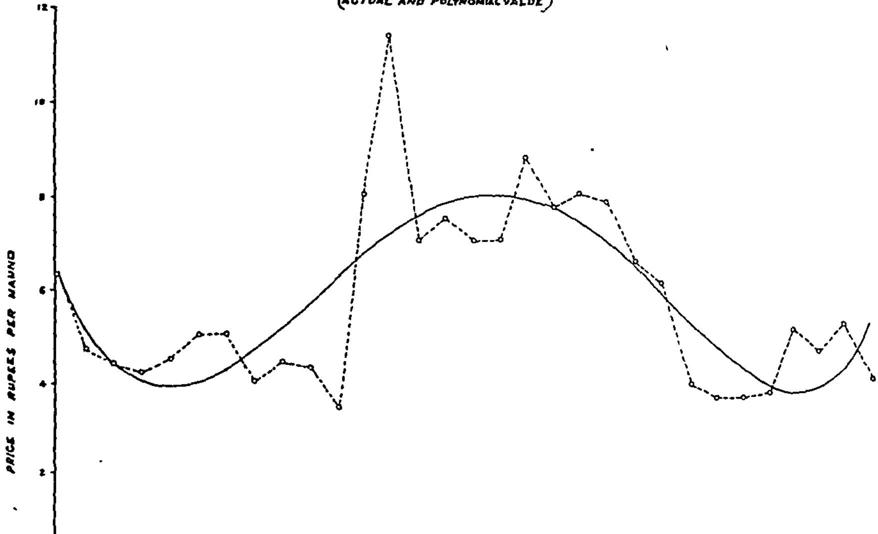


FIG. 217
PRICE OF RAPE-SEED AT HARVEST TIME
IN THE YEAR ENDING 15TH JUNE
(ACTUAL AND POLYNOMIAL VALUE)



·66, and ·12, respectively. These are all significant and show that the economic factors in general affect the area irrigated. If the progressive trend be eliminated, the correlations with the prices of cotton, wheat and rapeseed are —·26, ·26 and —·03, respectively. None of these is significant on 5 per cent level.

Correlation Coefficients.

(i) *Area irrigated during Kharif*—As seen from Table LVIII, the highest correlation of area irrigated during kharif with any single variable is that with the price of cotton at previous harvest. The regression formula is :

y — y (poly.) = 7392 (c—c poly.) .. .. (1)

where y = Area irrigated during kharif in acres.

y (poly.) = The polynomial value or the progressive change of y.

c = Price of cotton at Lyallpur at previous harvest in rupees per maund.

c poly. = Polynomial value of c.

The other significant correlation is that with the discharge utilized during kharif. The multiple correlation of area irrigated with the price of cotton and discharge utilised is :

y—y poly. = 6143 (c—c poly.) + 6400 (d—d poly.) .. (2)
where d is the discharge utilized in lakhs of cusec-days.

The values of area irrigated, calculated from equations (1) and (2) are plotted in Fig. 211 and are tabulated in Table LX.

The correlations with rainfall are low and not significant.

(ii) *Area irrigated during rabi.*—The correlations are given in Table LIX. The only significant correlation is with discharge utilized. Its value is ·74. The regression equation is :

y—y poly. = 50420 (d—d poly.) .. ..(3)

where y = Area irrigated during rabi in acres.

d = Discharge utilized during rabi in lakhs of cusec-days.

The values of area irrigated, calculated from equation (3) are plotted in Fig. 214 and given in Table LXI.

Of the non-significant correlations, the highest are with rainfall during October to January and with price of wheat. Both are positive and their values are ·35 and ·26 respectively.

Conclusions.—(i) The area irrigated during kharif shows a progressive trend which appears to be mainly due to the development of irrigation and improved distribution of water.

(ii) Apart from the progressive trend, the area irrigated during kharif is significantly correlated with the price of cotton and to a lesser extent with the discharge utilized.

An increase, above the polynomial value, of one rupee per maund in the price of cotton or of one lakh cusec-days in the discharge utilized has the effect of increasing the area irrigated by 6,4 es, on an average.

(*iii*) Rainfall does not affect the area irrigated ; kharif.

(*iv*) The economic causes are mostly responsible for the progressive change in the area irrigated during rabi.

(*v*) The departures from the progressive trend, of area irrigated during rabi, are highly correlated with the discharge utilised. An increase, above the polynomial value, of one cusec-day in the discharge utilized increases the area-irrigated by half an acre, on the average.

(*vi*) Apart from the effect of prices in general, the price of rapeseed does not affect the area irrigated.

The price of wheat and rainfall during October-January, though suggestive of the possibility of a real effect cannot be judged significant from the data.

LAYOUT OF AN EXPERIMENT AT JARANWALA FARM

During the course of reclamation, it was observed by the Land Reclamation Officer that a crop of American cotton, grown on land,
other leguminous crop,
of bolls), whereas tirak
From this it was considered that the attack of tirak was associated with the presence of salts within 6 feet from natural surface.

Layout of an experimental farm at Jaranwala was worked out to study the effect of artificial injection of salts in various concentrations on the American cotton plant.

ESTIMATING THE THICKNESS OF A FLOOR AT VARIOUS POINTS ON THE PROFILE OF A WORK.

The theory as given in the Central Board of Irrigation Publication No. 12 was explained in a note sent to Mr. J. P. Jain, Assistant Research Officer, United Provinces at the latter's request. This was illustrated by a case refered to the Institute by him.

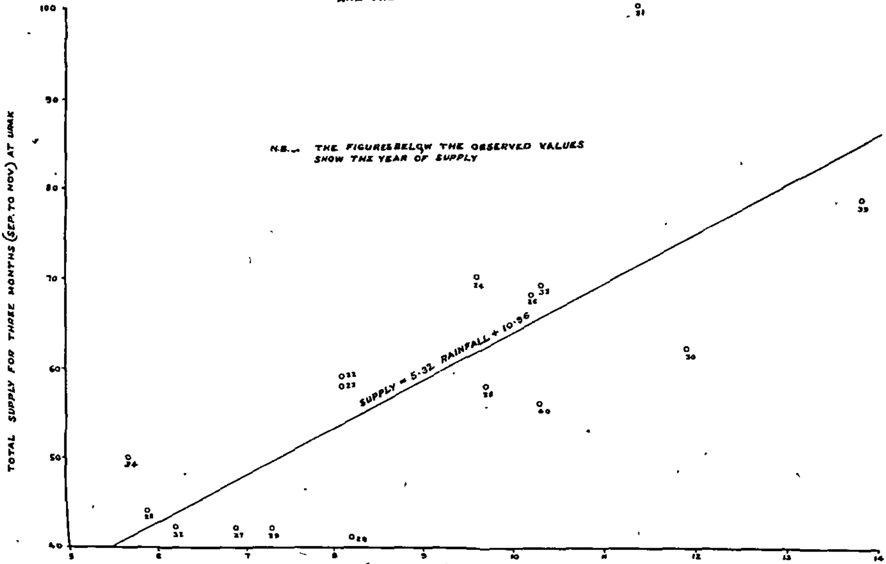
The effect of curvature of the glacis was also discussed. It was pointed out that in most cases, the reduction in the thickness of the floor due to curvature was likely to be small. In designing a work it would, therefore, be wise not to allow for it. This might be an error, but it was an error on the side of safety.

CORRELATION BETWEEN RAINFALL AT QUETTA AND WATER-SUPPLY AT URAK.

This was carried out at the request of Mr. E. R. Gee of the Geological Survey of India. Mr. Gee who had surveyed the area of the Urak catchment was of the opinion that the strata involved did not

FIG. 218

CORRELATION BETWEEN THE RAINFALL AT QUETTA AND THE WATER SUPPLY AT URAK



justify such a long-time lag between the rainfall of the catchment and the discharge figures as was suggested by the Baluchistan authorities.

The only data available with him was a diagram showing the monthly rainfall at Quetta and the monthly supply at Urak since 1921.

It appeared from a study of the diagram that this supply was a minimum during the months of September, October and November. It was because the rainfall during these months was practically *nil* and the main source of supply was the underground springs. It was decided in consultation with Mr. Gee to correlate the total supply during these months with the preceding September to August rainfall.

A straight line expressing supply in terms of rainfall was obtained by the method of 'least-squares'. It is plotted in Fig. 218 and the values calculated from it together with the percentage deviations from the actual values are given in Table LXII.

The correlation coefficient was found to be ·77. Taking account of the fact that Quetta is situated outside the catchment area, the correlation appeared to be quite high.

As most of the rain fell in the months of December to April, it was concluded that the time lag between the rainfall in Urak catchment and the discharge due to underground sources did not exceed 11 months and was possibly not less than 5 months. The exact time lag could not be worked out unless the daily rainfall figures in the catchment area and the contributions to the daily discharges made by the surface and underground sources were available.

FIG. 218

CORRELATION BETWEEN THE RAINFALL AT QUETTA AND THE WATER SUPPLY AT URAK

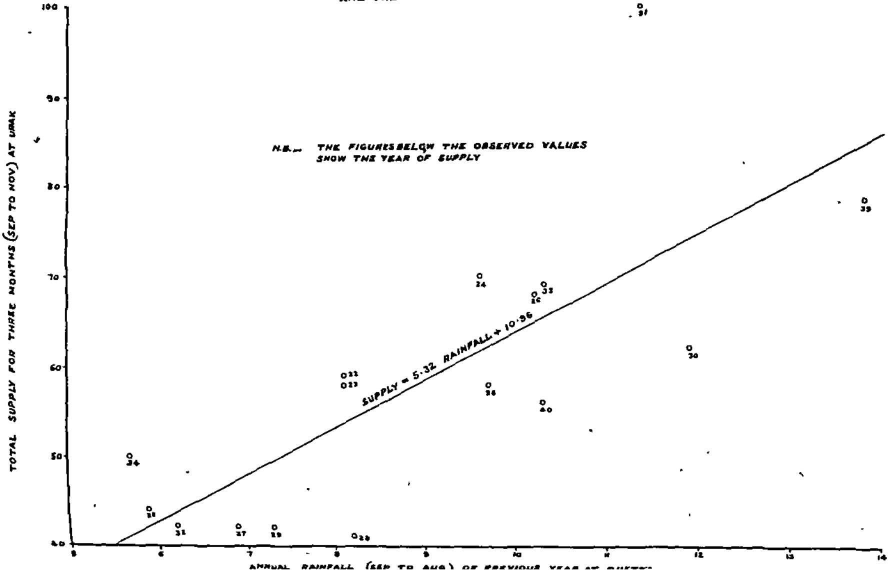


TABLE LIII

TABLE SHOWING THE AVERAGES OF THE VALUES GIVEN IN TABLE LII

1	2	3	4	5	6	7	8	9	10	11	12
Channel	R D.	M. V. P.	S D. of M. V. P.	M. V.	V.7D °c.	Difference of columns (5) and (6) as percentage	M. S. P.	S. D. of M. S. P.	M. S.	S.5D °c.	Difference of columns (10) and (11) as percentage
Lower Gugera Branch ..	6,000	·64	·102	2·65	2·57	3	·39	·118	·170	·104	11
		·67*	·075*	2·66*	2·62*	2*	·43*	·055*	·173*	·185*	7*
Burala Branch ..	6,000	·64	·058	2·53	2·40	5	·53	·107	·185*	·154	1
Mian Ali Branch ..	95,000	·76	·012	2·12	2·22	5	·57	·066	·233	·209	10
Shahkot Distributary ..	12,000	·73	·021	1·84	1·88	2	·43	·055	·240	·269	12
Khurrianwala Distributary ..	5,000	·70	·028	2·06	2·05	0	·51	·028	·104	·100	2
Mean for all channels ..	.	·69	·058	..	..	..	·49	·082	..	..	..
		·70*	·047*	.	..	..	·50*	·065*	..	..	..

Notation—

(1) S. D. = Standard deviation.

(2) The other notation used is the same as in Table XXXVIII.

*Mean after omitting observations on Lower Gugera Branch for 3rd November 1939.

TABLE LII

TABLE SHOWING MEAN VELOCITY POINT, MEAN VELOCITY, MEAN SILT POINT AND MEAN SILT FOR 6 CHANNELS

Date	M. V. P.	M. V.	V .7D c.	M. S. P.	M. S.	S .5D c.	Notation.
LOWER GUGERA BRANCH R. D. 6,000							
3rd November 1939..	·67	2·70	2·66	·37	·133	·150	M. V. P.=Mean velocity point on the central vertical.
4th November 1939..	·61	2·65	2·52	·40	·110	·120	
5th November 1939..	·48	2·60	2·28	·17	·204	·260	
6th November 1939..	·71	2·63	2·64	·42	·215	·238	
8th November 1939..	·59	2·71	2·60	·46	·187	·200	M. V.=Mean velocity.
9th November 1939..	·77	2·60	2·70	·52	·222	·217	
BURALA BRANCH R. D. 6,000							
18th November 1939	·59	2·44	2·21	·57	·101	·082	V .7D c.=Velocity at ·7D on the central vertical.
19th November 1939	·62	2·43	2·32	·70	·130	·108	
21st November 1939	·66	2·46	2·35	·59	·098	·078	
23rd November 1939	·58	2·60	2·29	·42	·191	·234	M. S. P.=Mean silt point on the central vertical.
2nd December 1939..	·74	2·65	2·74	·49	·203	·208	
4th December 1939..	·63	2·58	2·47	·43	·189	·215	
MIAN ALI BRANCH R. D. 95,000							
13th December 1939	·71	2·17	2·20	·60	·235	·198	M. S.=Mean silt intensity in gms/litre.
15th December 1939	·75	2·06	2·18	·52	·172	·167	
26th December 1939	·79	2·31	2·45	·51	·335	·330	S .5D c.=Silt intensity at ·5D on the central vertical.
28th December 1939	·81	2·19	2·35	·64	·255	·197	
9th January 1940 ..	·78	2·06	2·18	·65	·212	·180	
11th January 1940 ..	·71	1·91	1·93	·51	·187	·182	
SHAHKOT DISTRIBUTARY R. D. 12,000							
15th January 1940 ..	·72	1·86	1·91	·48	·237	·244	
19th January 1940 ..	·75	1·90	2·00	·45	·290	·320	
2nd February 1940	·72	1·69	1·71	·33	·190	·262	
4th February 1940 ..	·69	1·84	1·82	·48	·253	·261	
8th February 1940 ..	·73	1·86	1·91	·45	·232	·250	
11th February 1940	·74	1·89	1·95	·41	·239	·275	
KHURRIANWALA DISTRIBUTARY R. D. 5,000							
14th December 1939	·72	2·10	2·11	·51	·175	·172	
16th December 1939	·65	2·01	1·92	·51	·145	·140	
27th December 1939	·71	2·12	2·13	·55	·227	·210	
29th December 1939	·67	2·04	2·00	·53	·193	·180	
10th January 1940 ..	·72	2·07	2·10	·47	·208	·220	
12th January 1940 .	·71	2·04	2·06	·49	·214	·218	

TABLE LIII

TABLE SHOWING THE AVERAGES OF THE VALUES GIVEN IN TABLE LII

1	2	3	4	5	6	7	8	9	10	11	12
Channel	R D.	M. V P	S. D. of M. V. P.	M. V.	$V_{\cdot 7D}$ c.	Difference of columns (5) and (6) as percentage	M. S. P.	S. D. of M. S. P.	M. S.	$S_{\cdot 5D}$ c.	Difference of columns (10) and (11) as percentage
Lower Gugera Branch ..	6,000	·64	·102	2·65	2·57	3	·39	·118	·179	·198	11
		·67*	·075*	2·66*	2·62*	2*	·43*	·055*	·173*	·185*	7*
Burala Branch ..	6,000	64	·058	2·33	2·10	5	·53	·107	·185*	·154	1
Mian Ali Branch ..	95,000	·76	·042	2·12	2·22	5	·57	·066	·233	·209	10
Shahkot Distributary ..	12,000	·73	·021	1·84	1·88	2	·43	·055	·240	·269	12
Khurrianwala Distributary ..	5,000	·70	·028	2·06	2·05	0	·51	·028	·194	·190	2
Mean for all channels ..		·69	·058	..	..	..	·49	·082	..	..	..
		·70*	·047*	..	..	..	·50*	·068*	..	..	..

Notation—

(1) S. D. =Standard deviation.

(2) The other notation used is the same as in Table XXXVIII.

*Mean after omitting observations on Lower Gugera Branch for 3rd November 1930.

TABLE LIV

TABLE SHOWING THE PERCENTAGE DEVIATIONS OF $P/\sqrt{Q}$ AND $W/\sqrt{Q}$ FROM THEIR MEAN VALUES FOR 17 PUNJAB CHANNELS.

Section	Channel.	R. D.	Q.	P.	W.	$P/\sqrt{Q}$	$W/\sqrt{Q}$	Percentage Deviation from the mean values of $P/\sqrt{Q}$	Percentage Deviation from the mean values of $W/\sqrt{Q}$
Nanuana ..	Jhang Branch ..	7,260	2,823·94	147·54	142·01	2·776	2·689	+0·7	+4·0
	Rakh Branch ..	7,260	1,137·36	92·76	89·45	2·751	2·653	—0·2	+2·6
	Udhoki Distributary ..	2,372	37·98	19·23	17·65	3·120	2·864	+13·2	+10·8
Sagar ..	Main Line Lower ..	149,000	4,557·57	205·21	200·00	3·040	2·963	+10·3	+14·6
	Upper Gugera Branch ..	2,500	4,888·30	182·93	178·00	2·616	2·540	—5·1	—1·5
	Hafizabad Distributary ..	3,050	21·38	11·77	10·00	2·545	2·163	—7·7	—16·4
Pacca Dalla ..	Mian Ali Branch ..	95,000	402·14	53·95	50·00	2·691	2·494	—2·4	—3·6
	Khurrianwala Distributary ..	5,000	221·25	40·85	36·03	2·747	2·423	—0·4	—6·3
	Shahkot Distributary	12,000	163·57	31·93	32·25	2·731	2·522	—0·9	—2·5
Buchiana ..	Lower Gugera Branch ..	11,000	1,861·08	125·74	120·00	2·913	2·780	+5·7	+7·5
	Burala Branch ..	6,000	1,356·60	103·01	99·00	2·797	2·688	+1·5	+3·9
	Upper Gugera Branch	277,500	3,602·58	133·90	127·00	2·231	2·116	—19·1	—18·2
Uqbana ..	Rakh Branch I ..	213,880	587·27	65·23	62·00	2·692	2·559	—2·4	—1·0
	Rakh Branch II ..	235,620	421·98	59·08	55·80	2·876	2·717	+4·3	+5·1
	Lakhuana Distributary .	10,000	109·46	31·43	29·00	3·005	2·772	+9·0	+7·2
Tarkhani ..	Lower Gugera Branch ..	239,000	1,090·22	92·82	87·52	2·797	2·643	+1·5	+2·2
	Lower Gugera Branch ..	272,500	828·35	73·04	68·00	2·538	2·363	—7·9	—8·6

TABLE LV

TABLE SHOWING THE PERCENTAGE DEVIATIONS $V/(R^2 S)^{\frac{1}{3}}$ AND $V/(E R^2 S)^{\frac{1}{3}}$ FROM THEIR MEAN VALUES FOR 17 PUNJAB CHANNELS

Section	Channel	R. D	Q.	V.	R	$S\times10^3$	$E=P/W$	$\frac{V}{(R^2 S)^{\frac{1}{3}}}$	$\frac{V}{(E R^2 S)^{\frac{1}{3}}}$	Percentage deviation from the mean values of $V/(R^2 S)^{\frac{1}{3}}$	Percentage deviation from the mean values of $V/(ER^2 S)^{\frac{1}{3}}$
Nanuana ..	Jhang Branch ..	7,260	2,823·94	2·87	6·66	·224	1·03	13·35	13·22	−12·7	−11·6
	Rakh Branch ..	7,260	1,187·36	2·56	4·78	·213	1·04	15·11	14·01	−1·2	−0·3
	Udhoki Distributary ..	2,372	37·98	1·40	1·41	·203	1·09	16·76	16·29	+9·5	+8·9
Bagar ..	Main Line Lower ..	149,000	4,557·57	2·93	7·59	·201	1·03	12·95	12·82	−15·4	−14·3
	Upper Gugera Branch	2,500	4,888·30	3·04	8·78	·113	1·03	13·66	13·53	−10·7	−9·6
	Hafizabad Distributary	3,050	21·38	1·33	1·37	·271	1·18	16·66	15·77	+8·9	+5·4
Pacca Dalla ..	Mian Ali Branch .	95,000	402·14	2·20	3·39	·109	1·08	16·70	16·28	+9·2	+8·8
	Khurrianwala Distributary.	5,000	221·25	2·08	2·61	·223	1·13	18·09	17·37	+18·2	+16·1
	Shahkot Distributary	12,000	163·57	1·86	2·52	·232	1·08	16·35	15·93	+6·9	+6·5
Buchiana ..	Lower Gugera Branch	11,000	1,864·08	2·65	5·69	·231	1·05	13·71	13·49	−10·4	−9·8
	Burala Branch ..	6,000	1,356·60	2·56	5·12	·109	1·04	14·76	14·57	−3·5	−2·6
	Upper Gugera Branch	277,500	3,602·58	3·09	8·71	·196	1·05	12·57	12·36	−17·8	−17·4
Uqbana ..	Rakh Branch I .	213,880	587·27	2·25	4·00	·139	1·05	17·24	16·96	+12·7	+13·4
	Rakh Branch II ..	235,620	421·98	2·13	3·36	·166	1·06	17·28	16·94	+12·9	+13·2
	Lakhuana Distributary	10,000	109·46	1·70	2·04	·234	1·08	17·15	16·72	+12·1	+11·8
Tarkhani ..	Lower Gugera Branch	259,000	1,090·22	2·30	5·17	·175	1·06	13·75	13·49	−10·1	−9·8
	Lower Gugera Branch	272,500	828·35	2·33	4·86	·193	1·07	14·05	13·74	−8·2	−8·2

TABLE LVI

RUGOSITY COEFFICIENTS AND RELATED HYDRAULIC ELEMENTS FOR 42 REGULARLY OBSERVED PUNJAB SITES BELIEVED TO BE IN REGIME

(Q = Discharge in cu.ft./sec., V. = Velocity in ft./sec., R = Hydraulic Mean Radius in feet, S = Slope of water surface)
N_B = Barnes' N, N_K = Kutter's N, N_L = Lacey's N, N_M = Manning's N.

	2	3	4	5	6	7	8	9	10	11	12
	Name of channel	R. D.	Q.	V.	R.	$S \times 10^4$	N_B	N_K	N_L	N_M	P*
	Lower Chenab Canal System										
1	Main Line Upper ..	137,600	9,005·4	3·25	9·48	·19	·0292	·0294	·0308	·0282	9·2
2	Main Line Lower ..	149,000	4,463·4	2·91	7·57	·20	·0280	·0289	·0290	·0278	7·6
3	Main Line Lower ..	181,000	4,419·8	2·90	7·39	·20	·0282	·0284	·0294	·0275	6·9
4	Jhang Branch Upper ..	7,260	2,824·3	2·86	6·87	·21	·0279	·0280	·0289	·0272	6·2
5	Jhang Branch Upper ..	221,000	2,102·4	2·66	6·10	·18	·0255	·0256	·0263	·0250	5·2
6	Rakh Branch ..	7,260	1,080·9	2·54	4·72	·21	·0241	·0241	·0246	·0239	3·0
7	Mian Ali Branch ..	9,500	551·6	2·34	3·46	·22	·0216	·0216	·0216	·0215	0·5
8	Jhang Branch Lower ..	111,700	532·4	2·27	3·80	·23	·0243	·0243	·0245	·0243	1·2
9	Dhaular Distributary ..	12,000	289·6	2·16	2·80	·22	·0202	·0202	·0200	·0203	1·5
10	Nasrana Distributary ..	3,275	272·8	2·28	3·00	·17	·0176	·0176	·0175	·0177	1·1
11	Sarangwala Distributary ..	6,300	59·4	1·50	1·62	·26	·0215	·0213	·0208	·0220	5·8
12	Udhoki Distributary ..	2,500	40·4	1·53	1·50	·31	·0218	·0216	·0210	·0224	6·7
13	Khai Distributary ..	4,943	40·0	1·50	1·58	·29	·0223	·0221	·0215	·0229	6·5
14	Gugiana Distributary ..	7,345	29·0	1·33	1·34	·28	·0221	·0217	·0211	·0227	7·6
15	Jamal Jatti Distributary ..	2,300	27·3	1·31	1·20	·33	·0237	·0231	·0226	·0244	8·0
16	Hafizabad Distributary ..	3,050	22·5	1·40	1·36	·33	·0230	·0226	·0220	·0237	7·7

Lower Jhelum Canal System

17	Branch Lower	..	73,500	1,407·1	2·62	5·52	·17	·0244	·0215	·0251	·0240	1·0
18	Branch Lower	..	163,000	1,188·7	2·40	5·24	·13	·0208	·0208	·0213	·0205	7·9
19	Branch	..	102,500	784·5	2·15	4·48	·14	·0225	·0225	·0228	·0222	2·7
20	Branch	..	170,300	768·2	2·32	4·45	·16	·0221	·0221	·0225	·0219	2·7
21	Branch	..	204,200	735·8	2·25	4·51	·14	·0216	·0216	·0210	·0213	2·8
22	Branch	..	210,100	650·6	2·30	4·40	·15	·0218	·0218	·0221	·0215	2·8
23	Branch	..	227,000	611·4	2·27	4·10	·15	·0200	·0200	·0212	·0207	2·4
24	Branch	..	272,600	544·3	2·14	4·10	·15	·0222	·0222	·0224	·0220	1·8
25	Branch	..	281,500	493·6	2·14	3·78	·15	·0207	·0207	·0209	·0206	1·5
26	Distributary	..	2,675	440·3	2·12	3·44	·20	·0226	·0226	·0227	·0228	0·4
27	..stributary	..	1,350	361·5	2·12	3·70	·16	·0217	·0217	·0218	·0217	0·9
28	..nch	..	2,300	354·5	2·16	2·92	·20	·0108	·0108	·0107	·0109	1·0
29	Distributary	..	4,000	133·2	1·88	2·60	·20	·0221	·0221	·0210	·0222	1·4
30	..stributary	..	1,500	55·5	1·56	1·73	·28	·0225	·0223	·0218	·0230	5·5
31	Distributary	..	10,200	44·6	1·43	1·80	·22	·0224	·0225	·0217	·0228	5·1
32	Distributary	..	5,020	27·2	1·41	1·34	·31	·0219	·0216	·0209	·0226	8·1
33	..istributary	..	1,700	19·2	1·37	1·23	·27	·0205	·0202	·0193	·0211	8·2
34	..stributary	..	1,550	16·0	1·34	1·23	·27	·0202	·0200	·0193	·0209	8·3
35	i Distributary	..	7,000	7·8	1·00	·89	·34	·0223	·0215	·0209	·0233	11·5
36	Distributary	..	3,100	5·8	1·00	·79	·30	·0192	·0168	·0170	·0202	12·8

Lower Bari Doab Canal System

37	Main Line Lower	..	83,400	5,810·3	2·87	9·46	·12	·0263	·0266	·0277	·0254	9·1
38	Main Line Lower	..	115,000	5,676·9	2·84	9·37	·13	·0275	·0279	·0289	·0265	9·1
39	Jandwala Distributary	..	3,050	89·4	1·66	2·20	·17	·0199	·0198	·0195	·0201	3·1
40	Dhunlwala Distributary	..	18,500	42·3	1·40	1·66	·21	·0211	·0209	·0204	·0216	5·9

Upper Bari Doab Canal System

41	Alwal Distributary	..	1,500	80·1	1·72	1·79	·30	·0216	·0215	·0210	·0221	5·2

Sirhind Canal System

42	Akhara Distributary	..	9,050	17·9	1·51	1·22	·29	·0185	·0185	·0176	·0191	8·5

*$p = (N_L/N_M - 1)\,100$ if N_L is greater than N_M

$= (N_L/N_M - 1)\,100$ if N_L is less than N_M

TABLE LVII

TABLE SHOWING THE AVERAGE VALUES OF RUGOSITY COEFFICIENTS

Channel	R. D.	Number of observations	N_K	σ_K	N_M	σ_M	N_L	σ_L
Haveli Canal ..	80,000	16	·0193	·0019	·0186	·0019	·0196	·0019
Ditto ..	133,000	25	0147	·0008	·0146	·0008	·0157	·0008
Ditto ..	142,000	61	0135	·0015	·0137	·0015	·0140	·0016
Ditto ..	165,000	41	0171	0029	·0169	·0028	·0178	·0028
Ditto ..	220,000	57	·0157	·0015	·0160	·0015	·0174	·0017
Bikaner Canal ..	43,000	113	·0147	·0017	·0148	·0017	·0153	·0019
Ditto ..	188,000/ 188,764	41	·0143	·0007	·0143	·0009	·0147	·0010
Ditto	305,000	105	·0133	·0014	·0133	·0014	·0138	·0014
Ditto ..	343,000	47	·0143	·0022	·0143	·0021	·0149	·0021

N_K = Kutter's rugosity coefficient.

N_M = Manning's rugosity coefficient.

N_L = Lacey's rugosity coefficient.

TABLE LVIII

EA IRRIGATED ELIMINATION

Serial No.	Variables	Correlation Coefficient
1	Rainfall during April and May ..	·012
2	Rainfall during June and July ..	·008
3	Rainfall during August and September ..	—·184
4	Discharge utilized during kharif ..	·147*
5	Price of cotton at previous harvest ..	·680**
6	Discharge utilized during kharif and price of cotton at previous harvest.	·710**

*Indicates significance on 5 per cent level.
** Indicates significance on 1 per cent level.

TABLE LIX

TABLE SHOWING THE CORRELATION COEFFICIENT OF THE AREA IRRIGATED DURING RABI WITH THE FOLLOWING VARIABLES AFTER ELIMINATION OF PROGRESSIVE TREND

Serial No.	Variables	Correlation Coefficient
1	Rainfall during October and November	·193
2	Rainfall during December and January .	·227
3	Rainfall during February and March ..	—·055
4	Rainfall during October to January .	·347
5	Rainfall during October to March ..	·174
6	Discharge utilized during Rabi ..	·738**
7	Price of wheat at previous harvest ..	·264
8	Price of rapeseed at previous harvest ..	—·039

** Indicates significance on 1 per cent level.

TABLE LX

TABLE SHOWING THE VALUES OF THE AREA IRRIGATED DURING KHARIF, THE DISCHARGE UTILIZED, THE PRICE OF COTTON AND THE CALCULATED VALUES OF AREA IRRIGATED.

Year	Price of cotton (previous harvest) Rs. per maund		Discharge utilized during Kharif (cusec-days)		Area irrigated during Kharif (Acres)			
	Act.	Poly.	Act.	Poly.	Act	Poly	Cal. from formula (1)	Cal from formula (2)
1908-09	6·7	7·2	1,320,400	1,289,400	610,300	629,500	625,800	628,300
1909-10 .. .	5·9	6 6	1,297,000	1,378,500	674,900	659,100	654,000	640,400
1910-11 .. .	7·2	6·0	1,338,200	1,429,800	6[illegible]5,200	678,600	687,400	680,400
1911-12	10·0	6·1	1,562,600	1,454,200	710,000	691,500	720,000	723,400
1912-13	8·3	6·6	1,651,500	1,460,700	742,800	700,700	713,100	723,800
1913-14	8·5	7·5	1,469,900	1,456,700	718,800	708,600	715,900	715,800
1914-15	7·0	8·7	1,386,900	1,448,100	713,000	716,800	704,400	702,000
1915-16 .. .	4·5	9·8	1,438,400	1,439,400	652,100	726,500	687,700	692,400
1916-17	6·0	11·0	1,335,200	1,433,800	715,300	738,500	701,900	700,100
1917-18	10·0	12·1	1,054,900	1,433,400	727,300	753,200	737,800	716,500
1918-19	16·1	13·3	1,028,100	1,439,400	795,500	770,600	805,700	789,300
1919-20	21·8	14·1	1,478,200	1,462,000	851,200	790,400	846,700	841,500
1920-21 .. .	19·0	14·7	1,707,600	1,470,800	834,100	812,200	843,600	855,000

1921-22	9·5	15·1	1,571,500	1,494,600	854,400	835,300	794,300	804,300
1922-23	10·0	15·2	1,513,000	1,522,600	809,600	858,000	820,000	825,000
1923-24	12·0	15·1	1,369,800	1,552,500	828,500	882,300	850,600	850,700
1924-25	20·0	14·6	1,592,600	1,582,500	801,300	901,500	914,000	939,700
1925-26	18·5	14·1	1,652,100	1,610,700	975,400	924,500	957,000	955,000
1926-27	14·0	13·3	1,607,800	1,635,300	976,000	942,600	947,700	945,300
1927-28	9·0	12·3	1,632,300	1,654,800	934,100	957,200	933,100	934,600
1928-29	12·0	11·3	1,745,400	1,667,800	1,022,600	968,500	973,600	978,000
1929-30	11·0	10·1	1,569,500	1,673,800	993,800	976,300	982,000	975,400
1930-31	6·8	9·0	1,778,700	1,672,700	962,700	981,000	961,000	973,700
1931-32	4·9	7·8	1,728,600	1,665,300	931,100	984,300	962,100	968,800
1932-33	7·5	6·7	1,656,400	1,653,200	947,300	981,300	990,200	980,600
1933-34	6·2	5·7	1,405,400	1,635,100	972,800	985,600	989,300	971,100
1934-35	1·7	5·2	1,599,800	1,627,100	981,100	989,100	985,700	984,500
1935-36	5·5	4·7	1,681,100	1,612,300	1,028,500	998,400	1,004,300	1,007,300
1936-37	6·1	4·6	1,744,700	1,631,500	1,050,600	1,016,300	1,027,300	1,033,100
1937-38	7·1	5·0	1,734,900	1,663,100	1,092,000	1,047,000	1,062,400	1,065,100
1938-39	3·5	6·8	1,629,900	1,727,100	1,043,500	1,095,500	1,071,400	1,068,100

N. B.—Formula (1):—$Y - Y_{Poly.} = 7{,}312\ (C - C_{Poly})$.

Formula (2):—$Y - Y_{Poly.} = 6{,}413\ (C - C_{Poly.}) + \cdot 0640\ (d - d_{Poly})$.

Where—

Y = Area irrigated during Kharif in acres.
C = Price of cotton at previous harvest in Rs. per maund.
d = Discharge utilized in Kharif in cusec-days.
Poly. = Polynomial value of the variable.

TABLE LXI

TABLE SHOWING THE ACTUAL AND CALCULATED VALUES OF AREA IRRIGATED DURING RABI FROM THE DISCHARGE UTILIZED DURING RABI

Year	Discharge utilized during Rabi (cusec-days)		Area irrigated during Rabi (Acres)		
	Act.	Poly.	Act.	Poly.	Cal.
1908-09 ..	1,446,400	1,471,600	1,381,900	1,425,100	1,412,400
1909-10 ..	1,448,000	1,382,700	1,537,800	1,492,000	1,524,900
1910-11 ..	1,294,100	1,320,600	1,350,300	1,534,200	1,520,800
1911-12 ..	1,327,100	1,280,400	1,623,200	1,558,800	1,582,300
1912-13 ..	1,143,800	1,257,600	1,497,400	1,571,700	1,514,300
1913-14 ..	1,177,200	1,248,100	1,526,800	1,577,300	1,541,600
1914-15 ..	1,399,300	1,248,700	1,599,200	1,579,200	1,655,100
1915-16 ..	1,295,300	1,256,100	1,631,900	1,580,000	1,599,800
1916-17 ..	1,188,300	1,268,000	1,571,500	1,581,400	1,541,200
1917-18 ..	1,275,400	1,282,100	1,549,800	1,584,300	1,580,900
1918-19 ..	1,280,100	1,296,700	1,543,700	1,589,200	1,580,800
1919-20 ..	1,295,700	1,310,400	1,604,100	1,595,700	1,588,300
1920-21 ..	1,380,900	1,322,200	1,561,800	1,603,300	1,632,900
1921-22 ..	1,468,500	1,331,200	1,706,500	1,611,000	1,680,200
1922-23 ..	1,349,600	1,337,200	1,702,700	1,617,500	1,624,000
1923-24 ..	1,256,500	1,339,300	1,657,000	1,621,700	1,580,000
1924-25 ..	1,263,200	1,338,000	1,575,000	1,622,200	1,584,500
1925-26 ..	1,212,400	1,333,400	1,537,400	1,617,900	1,556,900
1926-27 ..	1,324,400	1,325,700	1,586,200	1,607,700	1,607,000
1927-28 ..	1,200,800	1,315,400	1,478,100	1,591,000	1,533,200
1928-29 ..	1,470,000	1,303,000	1,638,700	1,567,600	1,651,800
1929-30 ..	1,428,800	1,289,100	1,644,400	1,537,700	1,608,100
1930-31 ..	1,320,900	1,274,400	1,533,300	1,502,300	1,525,700
1931-32 ..	1,318,100	1,259,600	1,467,100	1,463,000	1,492,500
1932-33 ..	1,138,400	1,245,300	1,292,200	1,422,500	1,368,600
1933-34 ..	1,221,500	1,232,200	1,455,200	1,384,000	1,378,600
1934-35 ..	1,056,200	1,221,000	1,300,200	1,352,200	1,269,100
1935-36 ..	1,190,000	1,212,000	1,334,600	1,332,800	1,321,700
1936-37 ..	1,329,200	1,205,600	1,387,100	1,332,800	1,395,100
1937-38 ..	1,268,500	1,202,100	1,358,300	1,360,400	1,393,900
1938-39 ..	1,141,700	1,201,400	1,408,400	1,425,600	1,395,500

N. B.—

Formula : $Y - Y_{Poly.} = \cdot 5042\,(d - d_{Poly.})$

Where Y = Area irrigated during rabi in acres.

d = Discharge utilized during Rabi in cusec-days.

Poly. = Polynomial value of the variables.

TABLE LXII

CORRELATION BETWEEN THE RAINFALL AT QUETTA AND THE WATER SUPPLY AT URAK

Year	Annual rainfall at Quetta (September to August)	Total Supply at Urak during September to November of next year		$\left(1-\frac{\text{Col. 4}}{\text{Col. 3}}\right)\times 100$
		Actual	Calculated	
1	2	3	4	5
1921-22	8·1	59	54	8
1922-23	8·1	58	54	7
1923-24	9·6	70	62	11
1924-25	5·9	44	42	5
1925-26	10·2	68	65	4
1926-27	6·9	42	48	—14
1927-28	8·2	41*	55	—34
1928-29	7·3	42	50	—19
1929-30	11·9	62	74	—19
1930-31	11·4	100	72	28
1931-32	6·2	42	44	—5
1932-33	10·4	69	66	4
1933-34	5·7	50	41	18
1934-35	13·4	Not available	..	..
1935-36	8·0	..	..	..
1936-37	13·1	..	..	..
1937-38	9·7	58	63	—9
1938-39	13·8	78	84	—8
1939-40	10·2	56	65	—16

*This value is the total of supply in September and October multiplied by 3/2. The value for November was rejected as this was very much different from the supply in the other two months.

PLAN OF JARANWALA FARM

SHOWING THE LAYOUT OF PLOTS AND THE CROPPING SYSTEMS

SCALE 1" = 20 KARAMS

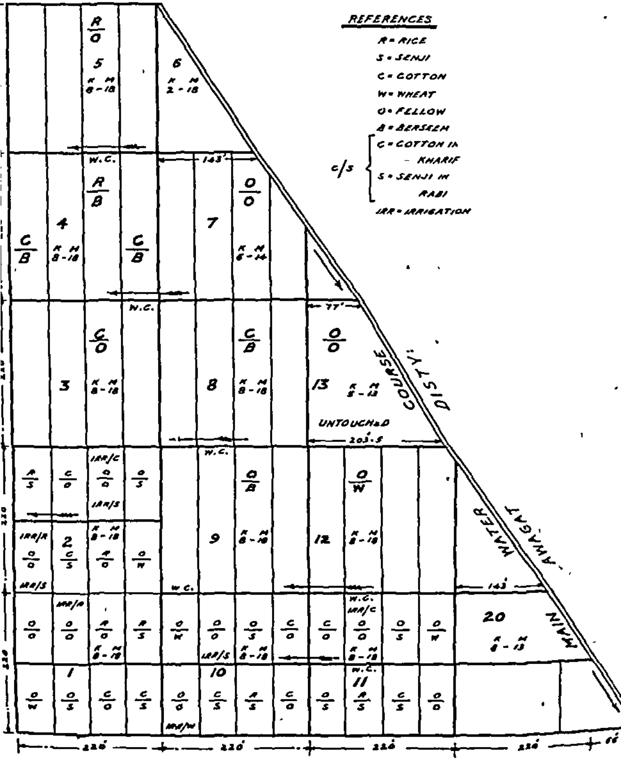


FIG. 219

JARANWALA EXPERIMENTAL STATION

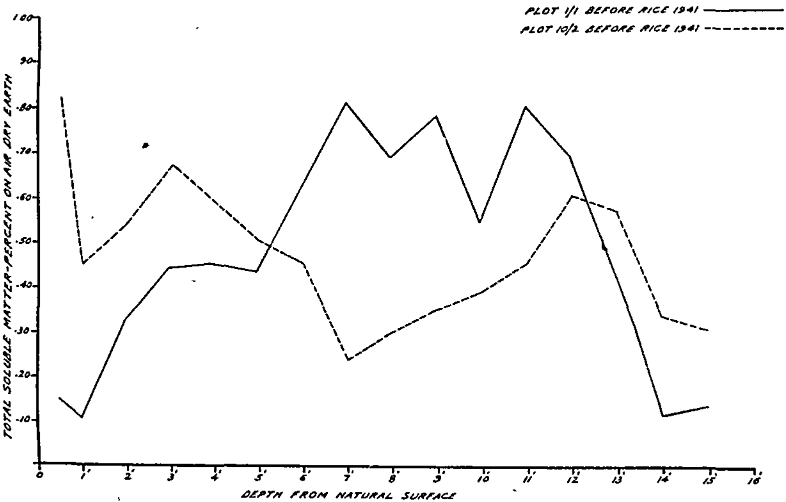


FIG.220

PLOT. 4

JARANWALA EXPERIMENTAL STATION

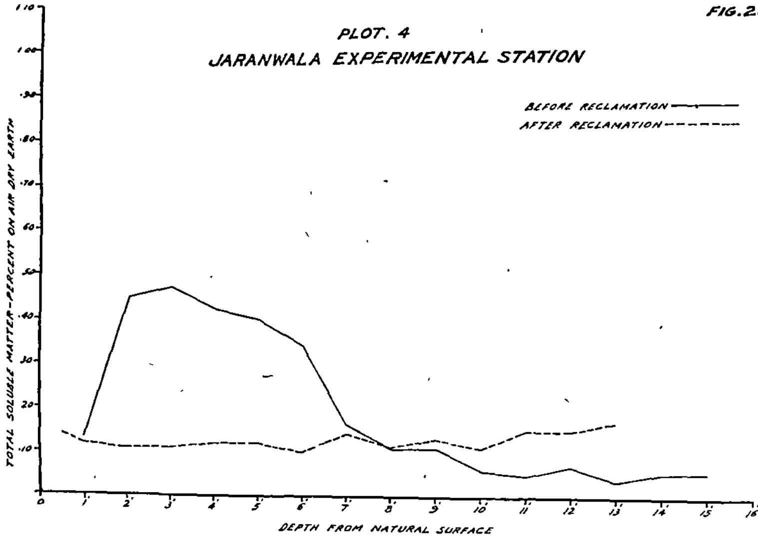


FIG. 221

PLOT. 5

JARANWALA EXPERIMENTAL STATION

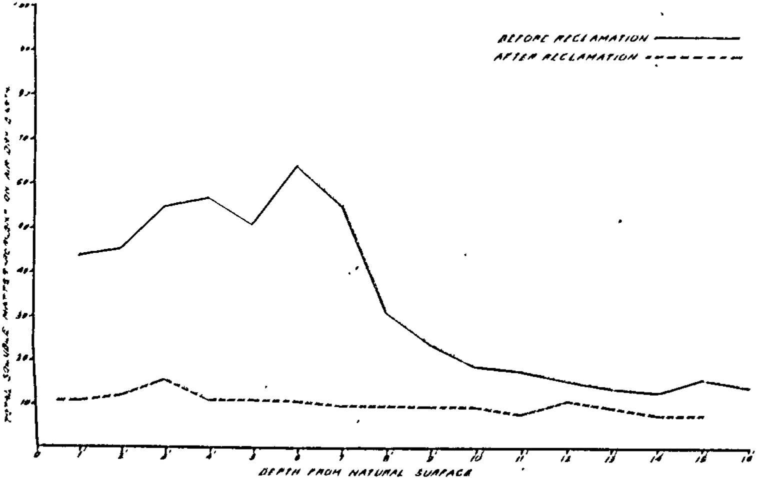


FIG. 222

PLOT 1/1
JARANWALA EXPERIMENTAL STATION

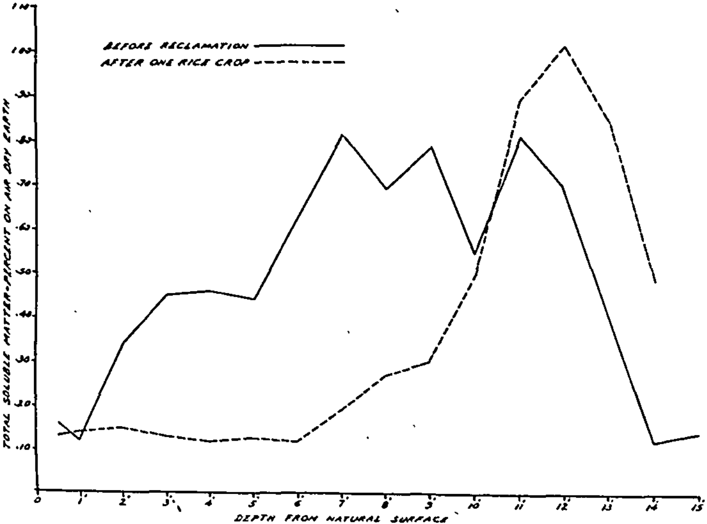
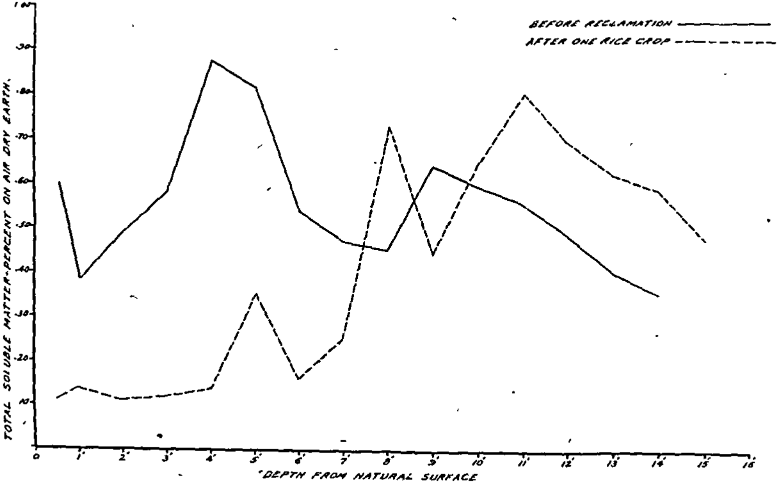


FIG. 223

PLOT 1/5

JARANWALA EXPERIMENTAL STATION



LAND RECLAMATION SECTION

EXPERIMENTAL

The Jaranwala Experimental Station

In the report for the year ending April 1940, a detailed description of the Jaranwala Experimental Farm was given. The experiments carried out at that Farm have demonstrated that—

(1) With the present duty of water, the normal crop rotations in undeteriorated canal colony areas lead to salt movement towards the surface. This was illustrated by photographs in the last annual report. Since then results of analyses have become available. Typical examples are given in Fig. 219. In plot 1/1 the main zone of salt accumulation occurred between 5 and 13 feet from the natural surface although due to the upward movement the salt content of the soil had exceeded the deterioration limit in the depth between the second and the fifth feet also. In the case of plot 10/2 the salt zone had appeared at the surface and the accumulation zone was confined to the first 6 feet. As the crop rotation in both the plots was the same the difference in the upward movement of the salt in the two plots is due to the initial depth at which the zone of salt accumulation was situated. It was originally nearer the surface in plot 10/2 than in 1/1. This has a bearing on the number of rice crops required for reclamation as is discussed later in this report.

(2) The growth of rice during the summer or of berseem during the winter on the normal quantity of water required for these crops either depresses the zone of salt accumulation or checks its rise to the surface.

Plots 4 and 5 are examples of this rotation. The results of analyses are represented in Figs. 220 and 221. It will be seen that with two rice crops the whole of the salt from the surface is removed to the water-table and although three cotton crops have been taken after rice the analyses indicate that the salt, once it is removed to the water-table, does not rise again towards the surface. During the last three kharif harvests American cotton gave yields of 17·5, 16·0 and 16·0 maunds per acre in one set of experiments and 16·25, 17·75 and 16·0 maunds in the second set.

During Kharif 1941 all fields where normal crops other than rice were grown and which had developed salt patches were put under rice. Soil samples have been taken after rice. The results of analyses are given in Figs. 222 and 223. These relate to plots 1/1 and 1/5. It will be seen that in the case of plot 1/1 where the concentration of the salt at the surface had not reached the maximum the removal of the salt from the soil profile is complete to a depth of 9 feet. In the case of plot 1/5 leaching is effective to a depth of 7 feet only. In order to free the soil profile of salt to a depth of 10 feet one more rice crop will be required.

In actual reclamation the introduction of a leguminous crop after rice is essential to set right the nitrogen balance of the soil which is upset on account of
rice. After rice, two t...
fields are carrying gram which has been sown in wadh-wattar of rice and which will be matured without irrigation while in others berseem has been sown with rabi irrigation. This will enable a study to be made of the differences in yields of normal crops grown after a barani crop of gram and an irrigated crop of berseem after rice.

188-N.B., *Lower Jhelum Canal*—A detailed description of this area and the history of its reclamation was given in the report for the year 1940. The reclaimed land continued to be on lease with a tenant. During Rabi 1940-41 wheat gave an average yield of 20·0 maunds per acre. In Kharif 1941, 4-F American cotton was sown in six fields. This gave an average yield of 11·0 maunds per acre. The condition of kharif crops of cotton and sugarcane of 1941 is illustrated in Figs. 224 and 225.

From the yields of crops for Rabi 1940-41 and Kharif 1941 it is concluded that, if thur land is completely reclaimed with one or two rice crops according to the initial position of the salt zone in the soil profile and the degree of alkalinit[illegible] yields of crops for a period of ten [illegible] of land under normal cropping on [illegible] to occur.

Reclamation of Thur Land—In Kharif 1941 the total area under reclamation was 8,500 acres. The total quantity of rice produced was approximately 100,000 maunds which added a sum of rupees three lakhs at least to the zamindars' income. It is expected that gram sown in wadh-wattar will give more than 50,000 maunds of grain which means a further addition of Rs. 1,50,000 to the zemindars' profits.

In 1941 only one rice crop was taken during the rice season instead of two. The sowing period was so arranged as to enable the zemindars to harvest their rice crop before the normal sowing period of gram. This was helpful in controlling the rice borer and enabling the zemindars to grow gram in wadh-wattar of rice. The absence of the attack of the rice borer has encouraged the zemindars and increased their profits.

Costing accounts have been maintained separately for each centre and for each site where reclamation was undertaken on extra water-supply at contract rates. Almost everywhere there has been a profit to the zemindars on the rice crop alone. If the income from the following rabi crop of gram is included, the profits increase considerably. The success of gram in wadh-watt[illegible] of rice is largely

FIG. 224

188 N. B.
COTTON KHARIF 1941
10 years after reclamation

FIG. 225

188 N. B.
SUGARCANE IN RECLAIMED LAND
10 years after reclamation

Fig. 23

188 N. B.

SUGARCANE IN RECLAIMED LAND

10 years after reclamation

FIG. 225

188 N. B.

SUGARCANE IN RECLAIMED LAND

10 years after reclamation

The progress of reclamation from 1940 is shown by the increase in the water-supplies allotted which are given in the following table :—

	Reclamation Centres for the benefit of petty zemindars		Reclamation on contract water-supplies for the benefit of big landlords	
	Number of Centres	Extra Water-supplies allotted	Number of Sites	Extra Water-supplies allotted
		Cusecs		Cusecs
1940	1	30	19	30
1941	3	137	44	76
1942	5	260	105	200

Sangla-cum-*Sukhcki Centre*—An account of the progress of reclamation in this Centre was given in the last annual report. It was stated that the average net income to the zemindar, after deducting all expenses, from the first crop of rice and the following rabi crops of gram, wheat and berseem was Rs. 36 per acre. The costing accounts for the area taken up in 1939 were continued. These accounts show that the yields of the second rice crop and the rabi crops following rice were higher than in the previous year and gave a net profit of Rs. 50 per acre. The total profit to the zemindar for the four harvests, therefore, amounts to Rs. 86 per acre from land which was lying thur and had become valueless.

On account of the profits realised and the assurance that reclamation will continue, some zemindars have started speculation in the sale and purchase of thur land in this area. In Kharif 1941 thur land unreclaimed was sold at Rs. 150 per acre.

In Rabi 1940-41 gram sown in wadh-wattar of rice matured without any irrigation and gave an average yield of 13·2 maunds per acre. The yield of 13·2 maunds, was higher than the yield of gram obtained from the undeteriorated land to which canal irrigation had been applied.

As a result of two successful crops of rice and the rabi crops of gram and berseem, 418 acres were handed back to the zemindars after reclamation for cultivation on their own water-supply. In this area the following kharif crops were sown :—

	Acres
Sugarcane	27
Cotton	121
Fodders	20·5

The remaining area was prepared for rabi crops. Sugarcane gave an average yield of 527 maunds of stripped cane per acre which is equivalent to 58 maunds of gur.

In reclaimed land at Sukheki the yield of the Desi variety of cotton was 7·2 maunds, while the American gave 10·8 maunds. In both cases the yield of cotton from reclaimed land is higher than that obtained from undeteriorated fields. In this locality cotton both in reclaimed land and in normal land has not been as successful as in the year 1940. This is probably due to seasonal factors. Fig. 226 shows a typical cotton field in reclaimed land.

In 1941, 13 villages were added to the Reclamation Centre. Rice was sown in 1,930 acres out of which 177 acres failed on account of high alkalinity and insufficient leaching. Rice was sold by zemindars at prices varying from Rs. 3 to Rs. 4-5-0 per maund. In calculating profits an average price of Rs. 3-12-0 per maund has been taken. A summary of the accounts for rice is given in Table LXIII. This shows a net profit to the zemindar of approximately Rs. 30·5 per acre.

A soil survey of the fields in which rice was sown in Kharif 1941 is in progress. It is expected that approximately 225 acres will be fit for normal cropping on zemindari water-supply. These fields will be handed back to zemindars and more thur included for reclamation.

A point worth mentioning is that, on account of the heavy rains in September 1941 the water-table rose and came within a foot of the surface in the depressions. Observations show that wherever leaching was in progress and rice had been sown fields came into wattar (fit condition for ploughing and sowing) for sowing rabi crops. Those, in which salt was present at or near the surface did not dry and remained too wet. The zemindars were unable to sow rabi crops. The wet condition of land is due to the presence of sodium sulphate which on account of its hygroscopic properties keeps the soil supplied with moisture and does not allow it to become dry.

Along the Vanir Branch Drain where the water-table is high the zemindars have, acting on the advice of the Department, started constructing field drains. So far 5,000 feet of field drains have been dug. The Executive Engineer, Rechna Drainage Division, has agreed to provide the outfalls for these drains. It is expected that all the field drains will be completed before Kharif 1942. Reclamation will be started when the outfalls have been provided. The water from the [illegible]. It contains 160 parts of total soluble [illegible]. When all field drains begin to func[illegible] expected that the salt from the soil crust will be washed into the field drains and then into the Vanir Branch Drain. This will enable reclamation and cultivation to be carried on.

The prosperity of the zemindars in the Sangla-*cum*-Sukheki Centre is evidenced from the transactions in land that have taken place during the year. As mentioned above thur land has been sold

FIG. 226

SUKHEKI-CUM-SANGLA RECLAMATION CENTRE

Cotton in Reclaimed Land

Kharif 1941

at a price of Rs. 150 per acre. In one case the zemindar of the adjoining field paid Rs. 500 for an area of 1·5 acres which was lying thur and which had been included for reclamation during Kharif 1942. Other zemindars have invested their earnings from thur land in purchasing land elsewhere.

Two New Centres—From the results of the thur girdawaris it was known that the areas near Nankana Sahib and Jaranwala were deteriorating rapidly. Three new Centres were proposed for Kharif 1941 out of which two were finally sanctioned. Nankana Sahib and Dangali areas were selected.

An idea of the rate of deterioration in these areas can be obtained from the thur figures given in Table LXIV which relate to 12 villages irrigated by the Upper Gugera Branch, Lower Chenab Canal. In the case of Mahmunwali for example the area under thur has increased from 9·8 per cent in 1935 to 30·5 per cent in 1940. In Feroze it has increased from 0·97 per cent in 1935 to 18·3 per cent in 1940. The figures given in the statement represent the areas in which the damage due to thur has exceeded 75 per cent.

In the Nankana Sahib and Dangali Reclamation Centres, Government had sanctioned, as relief to zemindars whose lands had become thur, temporary cultivation of undeteriorated Crown land either in the same village or in the adjoining villages and had under consideration several applications for permanent exchange. This made the task of selection of fields for reclamation difficult. The zemindars had no previous experience of land reclamation or rice cultivation and thought they would gain nothing from the cultivation of land which had developed salts at the surface. They also considered that, if they accepted the increased supply of water for reclamation their claims for exchange of land which had previously been admitted by Government would not stand. With the close co-operation of the Deputy Commissioners the majority of the zemindars agreed to start reclamation although their decisions in some cases were late. Zemindars of Chaks 561, 564, 566 and 587-G.B., however, did not agree and asked for the imdadi pipes, installed to give the extra water-supplies, to be removed. This decreased the area under rice.

Another difficulty was that when fields were prepared and leaching had to start the Superintending Engineer found it impossible to run the 10 cusecs originally contemplated in the Nankana Minor. The supply that could be allotted was only 1·77 cusecs.

Some of the zemindars had made efforts to reclaim their thur fields by scraping salts from the surface and either dumping them in a heap in a corner of the field (Fig. 227) or carting it to the nearest village road if the field happened to be near it. This method enabled the zemindars to obtain one or two crops after scraping. After a period of six months or a year the field became thur again. This would be expected since at least 100 tons of salt have to be removed from an acre of land before the soil profile can be free of the accumulated salt

FIG. 227

126 G. B.

LOWER CHENAB CANAL

The Mound in the Field is the Salt Earth Scraped from the field by the Zamindar

at a price of Rs. 150 per acre. In one case the zemindar of the adjoining field paid Rs. 500 for an area of 1·5 acres which was lying thur and which had been included for reclamation during Kharif 1942. Other zemindars have invested their earnings from thur land in purchasing land elsewhere.

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schedule of assessment applicable to reclamation centres was given in *the report for the year 1941.*

For the big landlords who can afford to bear the entire cost of reclamation Government decided to allot extra water wherever possible for reclamation at contract rates. The conditions are—

(*a*) that the landlord or a group of landlords who apply for reclamation facilities should possess on the same outlet not less than *250 acres of land;*

(*b*) that facilities for reclamation should be given for land which was previously under cultivation and which went out of cultivation on account of the appearance of thur;

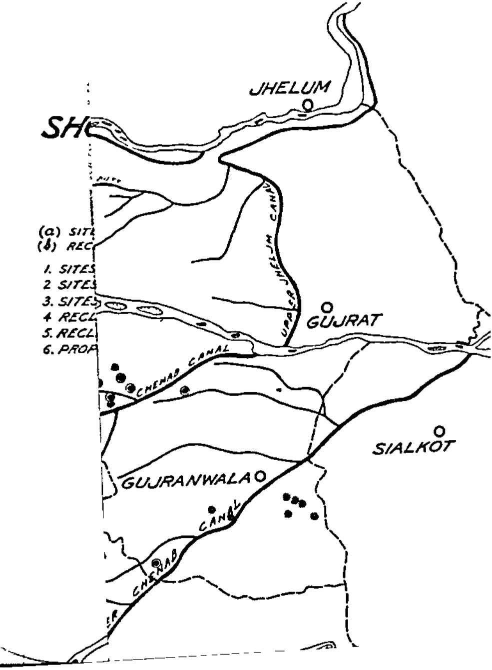
(*c*) that the charges for the extra water-supply should be the divisional rate per cusec;

(*d*) that for the supervision of reclamation a trained Village Assistant will be appointed whose pay, travelling allowance, etc., will be borne by the landlord.

As *already stated, in the year 1940 the big landlords utilised* only 30 cusecs of extra water-supply for the reclamation of deteriorated land in 19 estates. In 1941 the extra supply sanctioned was 76 cusecs for 44 sites and the applications considered by the Land Reclamation Board for 1942 are for 200 cusecs for 105 estates. Fig. 228 is a map *of the canal irrigated tracts of the Punjab in which localities where* reclamation has been in progress are marked.

The progress of reclamation for each canal system as indicated by the extra water-supplies utilised, is given in the following table :—

Serial No.	Canal	Extra water-supplies for reclamation		
		Utilised during		*Sanctioned for*
		Kharif 1940	Kharif 1941	Kharif 1942
		Cusecs	Cusecs	Cusecs
1	Lower Bari Doab Canal ..	15·5	34·5	70·4
2	Lower Chenab Canal East and West Circles	7·5	33·25	80·75
3	Upper Chenab Canal	0·5	2·5	24·0
4	Lower Jhelum Canal	4·0	3·5	20·0
5	Upper Bari Doab Canal ..	2·5	2·5	2·86
6	Sutlej Valley Project	0·0	0·0	2·0
	Total ..	30·0	76·25	200·01



the borer attack w

(a) Lower Bari Doab Canal.

Renala Estate—The yearly survey of the Renala Estate area was carried out in January 1941. The total area under thur was 1,489 acres as compared with 1,276 acres in 1940, i.e., new thur had appeared in an area of approximately 213 acres.

In the Renala Estate reclamation has been in progress in two different blocks, viz., Block A and Block B. Block A is situated in a depression where the water-table is within 5 to 8 feet of the natural surface. During the rice season it rises to within two feet. Reclamation in this block has been treated as an experiment. In Kharif 1941, rice was sown in 246 acres in this block. The yield of rice varied from 12·6 to 16·5 maunds, per acre. In land reclaimed in previous years cotton was sown in 109 acres, sugarcane in 20 acres and fodder in 12 acres. Cotton gave an average yield of 7 maunds per acre. Seven maunds of cotton per acre from reclaimed land is a low yield. This is due to the fact that cotton fields are situated in between rice fields. As there was more rice than cotton in this block, the water-table during the cotton growing season remained very near the surface. This depressed the yield of cotton. In Kharif 1942 reclamation will be undertaken in another block. It is expected that the yield of cotton in 1942 will increase as there will be no rice cultivation in the block. The condition of kharif fodder crops was excellent. 5·24 acres sugarcane which had been sown late gave an average yield of 42·2 maunds of gur. The rest was sold standing.

During Rabi 1941-42 the following crops have been sown :—

	Acres
Gram	150
Wheat	16
Berseem	122
Turnips	24

The condition of the rabi crops is good.

A portion of the extra water-supply sanctioned for experimental work in the Renala Estate has been utilised in Block B which is situated above the dhaya and where the water-table is 16 feet from the natural surface. Reclamation in Block B has been in progress since 1939. In Kharif 1941 rice was sown in 33·5 acres. The average yield of the jhona variety exceeded 16 maunds per acre whereas Sathra yielded only 8 maunds. The poor yield of Sathra is due to the fact that when the rice borer attack was noticed, the Jhona crop was replaced by Sathra by raising fresh nursery. The crop was sown late and was harvested late. This reduced the yield but the borer attack was controlled

In reclaimed land in Block B cotton was sown in 11 acres and kharif fodders in 12·5 acres. Cotton gave an average yield of 8 maunds and kharif fodder approximately 250 maunds per acre. In Rabi 1941-42, the following crops have been sown in reclaimed and partially reclaimed land of Block B :—

	Acres
Gram	50·25
Wheat	3·25
Berseem	4·5

The condition of the standing crops is good.

Costing accounts for reclamation in Blocks A and B were given in the report for the year 1940. These accounts were continued and have been brought up-to-date by Colonel Bruce, Agent, Renala Estate. A summary of the accounts is given in Table LXVII. These accounts show a net profit of Rs. 18,172 to the estate.

Three hundred and fifty acres of reclaimed land have been handed over to the estate for normal cropping on zemindari water-supply.

In addition to the experimental reclamation in Blocks A and B the Agent, Renala Estate, applied for two cusecs of water at contract rates for the prevention of land deterioration in areas where signs of thur were visible at the surface. The two cusecs of extra water-supply were utilised in a new block called C and 98 acres of rice were sown. This gave a yield of approximately 13 maunds per acre. During Rabi 1941-42 the following crops have been sown after rice :—

	Acres
Gram	76
Berseem	21·75

The condition of standing crops of gram and berseem is uniform and good. The thur patches have disappeared. A soil survey of this area is in progress. The Agent, Renala Estate, has applied for permission to extend rice cultivation to other areas instead of taking a second rice crop in Block C. This will be decided when the results of analyses are available.

Colonel Bruce has prepared a scheme for the cultivation of rice in the remaining areas of the estate to over-take deterioration as quickly as possible. He has applied for an additional supply of 3·5 cusecs at contract rates.

Chak 21-D belonging to Pir Mohammad Hussain.—A description of reclamation in this chak was given in the report for the year 1940. From the results of analyses after two rice crops it was known that the salt was at a depth of about 7 to 8 feet from the natural surface. For complete reclamation a third rice crop was necessary. This was sown

in Kharif 1941. Costing accounts for 50 acres of thur land taken up for reclamation in 1939 are given below :—

	Kharif 1939	Rabi 1939-40	Kharif 1940	Rabi 1940-41	Total	Profit
	Rs	Rs	Rs.	Rs.	Rs.	Rs.
Landlord's Share						
Income	310	558	539	941	2,348	..
Expenditure	545	38	554	48	1,185	1,163
Tenants' Share						
Income	310	558	455	731	2,054	..
Expenditure	178	38	188	185	589	1,465

For the four harvests the landlord has had a profit of Rs. 1,163 whereas the profit to the tenants was Rs. 1,465. After three rice crops the land has been completely reclaimed. Final costing accounts will be available in the next year's report.

During Kharif 1941 the Pir Sahib extended reclamation to the adjoining area on a further supply of one cusec from the Shergarh distributary.

Chak 97/9-L—This chak which consists of an area of 1,100 acres belongs to Rai Bahadur Basakha Singh. Approximately 250 acres had remained untouched until 1938 when reclamation was started. As the land was old kallar and was of the Bara type the landlord started reclamation on direct labour under the supervision of the Land Reclamation Department. In 1938 reclamation was undertaken on ·5 cusec of extra water-supply in an area which was more thur than rakkar. The Rai Bahadur applied for extension of reclamation to the Rakkar type. The progress of reclamation is given below :—

		Extra water-supply	Area under rice
		Cusecs	Acres
Kharif 1938	..	·5	29
" 1939	..	·5	30
" 1940		2·0	92
" 1941	..	3·0	131

After two rice crops American cotton was tried in Kharif 1941 in 8 acres, sugarcane in half an acre, Desi cotton in 2 acres and a Kharif crop of Guara in 7·5 acres. American cotton gave an average yield of 11·7 maunds per acre. In Rabi 1941-42 the following crops have been sown :—

	Acres
Gram	51
Wheat	4
Berseem	11
Senji and Shaftal	53

In reclaimed land in Block B cotton was sown in 11 acres and kharif fodders, i[illegible] of 8 maunds and khar[illegible]re. In Rabi 1941-42, [illegible] and partially reclaimed land of Block B :—

	Acres
Gram	50·25
Wheat	3·25
Berseem	4·5

The condition of the standing crops is good.

Costing accounts for reclamation in Blocks A and B were given in the report for the year 1940. These accounts were continued and have been brought up-to-date by Colonel Bruce, Agent Renala Estate. A summary of the accounts is given in Table LXVII. These accounts show a net profit of Rs. 18,172 to the estate.

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	Acres
Gram	76
Berseem	21·75

The condition of standing crops of gram and berseem is uniform and good. The thur patches have disappeared. A soil survey of this area is in progress. The Agent, Renala Estate, has applied for permission to extend rice cultivation to other areas instead of taking a second rice crop in Block C. This will be decided when the results of analyses are available.

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EXPENDITURE

No.	Particulars	Landlord's share	Tenants' share	Total
		Rs. A. P.	Rs. A. P.	Rs. A. P.
1	Land revenue on above area	253 11 6	253 14 6	507 13 0
2	Water rate for—			
	Wheat ..	2 2 0	2 2 0	4 4 0
	Berseem ..	25 15 0	25 15 0	51 14 0
3	26 maunds gram seed at Rs. 2-12-0	..	71 8 0	71 8 0
4	4·2 maunds berseem seed at Rs. 15 per maund	..	63 0 0	63 0 0
5	30 seers wheat seed ..	1 2 0	1 2 0	2 4 0
	Total ..	283 1 6	417 9 6	700 11 0

INCOME

Crops.	Total yield	Wages	Landlord's share	Tenants' share
	Mds. S.	Mds. S.	Mds. S.	Mds. S.
Gram ..	773 0	45 19	375 5	352 16
Wheat ..	8 7	0 19	3 39	3 29

Particulars	Landlord	Tenants
	Rs. A. P.	Rs. A. P.
Price of gram at Rs. 3-12-0 per maund	1,406 12 3	1,221 6 0
Price of wheat at Rs. 3-5-9 per maund	13 6 0	12 9 0
Price of berseem	260 15 9	260 15 9
Total ..	1,681 2 0	1,[illegible] 0 9

During Rabi 1941-42 the following crops have been sown :—

	Acres
Gram	36·62
Berseem	5

From the costing accounts, the total net profit to the landlord as a result of two kharif crops of rice and a rabi crop in between has been Rs. 871. The condition of the land has considerably improved. This is what the landlord values more than the actual profit. A soil survey is in progress. It is expected that after two rice crops some of the area will be handed back to the landlord for cultivation on zemindari water-supply.

Chak 25/11-*L belonging to Rai Sahib Chaudhri Bahadur Chand*— After a successful crop of rice in Kharif 1940, gram was sown in 16 acres and berseem in another 16 acres. Through the carelessness of the tenants gram was not sown in time. This gave a poor yield. The crop of berseem was normal. In Kharif 1941 approximately 49 acres of rice were sown. A costing account has been maintained since Kharif 1941. According to this account the total expenditure incurred by the landlord is Rs. 538. The total income to the landlord is Rs. 803 which means a net profit of Rs. 265. In a portion of the reclamation area the soil crust is thin and the sandy stratum starts from a depth of about 3 to 4 feet. A soil survey is in progress. It is expected that at least half the area will be fit for normal cropping during Kharif 1942.

Chak 126/15-*L belonging to Sardar Bahadur Ujjal Singh, M.L.A.*— The total area of this village is 2,673 acres out of which 2,503 acres is culturable commanded. The entire land has been purchased by Sardar Bahadur Ujjal Singh. A large portion of this estate is Bara land of which both the salt content and the pH values are very high. For the reclamation of this area the landlord bought two tractors and steam tackle with the necessary implements. According to the Sardar Bahadur he had spent Rs. 3,56,941 on reclamation and he had been able to bring under cultivation only 1,635 acres in a period of 18 years. In addition to steam and tractor cultivation, the landlord spent large sums of money on carting silt to the fields. According to his accounts it cost him approximately Rs. 200 to reclaim one acre of land, the standard of reclamation being a yield of cotton varying from 3 to 8 maunds. The estate was noted for the tirak attack on cotton. The normal zemindari water-supply for this chak is 8·4 cusecs. From 1922 to 1934 the area irrigated was much below the permissible. Since 1934 cultivation was increased, but the yields of crops were never normal. This is supported by the kharaba given. It has varied between 11 and 24 per cent. The land is classed fourth class for land revenue assessment and is put in schedule B regarding water-rates.

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Berseem	5

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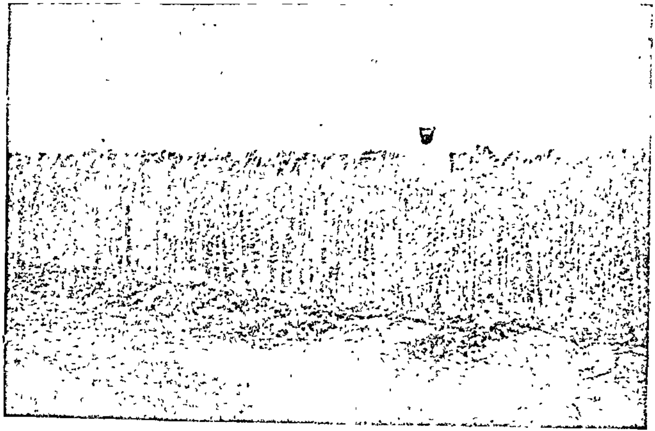
Fig. 229

L. B D. C. CHAK 126/15-L.
Rice in area under Reclamation
Kharif 1941



In Kharif 1940 the landlord applied for assistance. Reclamation was started in a compact block on an extra supply of two cusecs. From the soil survey of the area the Sardar Bahadur was informed that reclamation would take at least three rice crops and that the yields of rice in the first two years will be poor! A further difficulty was that the tenants and the beldars employed had no experience of rice cultivation. During Kharif 1940, 102 acres of rice were sown. Of this 26·5 acres were a complete failure. The area harvested gave an average yield of 7·3 maunds for the Sathra variety and 2·5 maunds for the Jhona variety. Gram and berseem following rice were more successful. Seeing the growth of gram and berseem the tenants at the farm applied for land to be given to them on batai. Although the land was only partially reclaimed the Sardar Bahadur was satisfied with the method of reclamation and applied to the Land Reclamation Board for an increase in the water-supply. It may be mentioned that the rate for the contract supply of water is higher than the rate in schedule B under which the land is ordinarily assessed. In spite of this, the landlord was willing to spend money on the extra water-supply and increase the rate of reclamation. In Kharif 1941 reclamation was extended in chak 126/15-L and also in land belonging to the Sardar Bahadur in other villages. The following are the details :—

——	Extra water-supply	Rice sown in acres
	Cusecs	
Chak 126/15-L ..	8 ..	342
„ 125/15-L ..	2 ..	94·5
„ 131/16-L ..	2 ..	81

of salt had been noticed at the surface. During Kharif 1940 only 133 acres of rice were sown which gave an average yield of 13 maunds per acre. It was stated that the conditions in Khanewal did not permit a larger area of rice to be matured. The experiment on reclamation in the B. C. G. A. Estate was, therefore, restricted to 3 cusecs in Kharif 1941 so that the area per cusec reclaimed could be determined. A new compact block of land was selected and a Research Assistant was posted to supervise reclamation. In the year 1941 it was possible to grow 123 acres of rice on 3 cusecs of extra water-supply which is equivalent to a duty of 41 acres. Out of the total area sown approximately 20 acres were flooded by salt-charged water from the adjoining salt land which was caused on account of a breach in the watercourse of a zemindar outside the estate. This damaged the rice crop. Excluding the area failed the average yield of rice during Kharif 1941 was approximately 11·5 maunds per acre. In the wadh-wattar of rice 90 acres of gram were sown. 19·8 acres of berseem and 13·1 acres of gram were sown after rauni. The condition of the standing rabi crops is good.

The area which carried rice in Kharif 1940 was also kept under observation. During Rabi 1940-41 gram was sown in 67 acres and berseem in 49 acres. Gram was matured barani without any irrigation and it gave a yield of over 14 maunds per acre against 8·3 maunds, the average yield of gram in the estate as a whole. From the field examination of soils and the yields of rice and following crops of berseem and gram it was indicated that a second crop of rice should be taken to depress the salt zone to a depth of more than 10 feet. As, according to the decision of the Land Reclamation Board, reclamation had to be shifted to a new compact block, the B. C. G. A. grew cotton according to their normal cropping scheme. From the yields of rice and the following rabi crops the area was divided into two classes. In class I were included those fields where the yield of cotton was expected to be normal and in class II those where the expected yield was below normal. Final yields for cotton are given below :—

	Area harvested in acres		Average yield per acre
			Mds.
Chak 85/10-R ..	Desi cotton	2·0	9·35
	American cotton..	48·03	8·96
Chak 81/10-R ..	American cotton..	35·67	11·98

The following is an extract from one of Mr. Jones' reports :—

"*On the whole I am inclined to agree that the cotton crop* in the poor land treated to reclamation is better than that on similar land nearby and I feel pretty sure that poor land treated to rice crop as per your scheme greatly benefits thereby and the attack of tirak is reduced."

Where the salt is heaviest your treatment undoubtedly is of very considerable benefit and we are convinced of this ".

As the area under reclamation in Kharif 1940 was abnormally low and a new block of land had to be selected for reclamation in Kharif 1941 costing accounts cannot be prepared. The area under reclamation in Kharif 1941 will carry rice again in Kharif 1942 if the salt zone is not depressed to a depth of 10 feet. At the end of Kharif 1942 correct accounts will be available. These will be given in the next annual report.

Chak 57/4-R belonging to Captain Sodhi Harnam Singh, M.L.A.—1·5 cusecs of extra water was sanctioned with effect from Kharif 1941 for the reclamation of salt land in this village. Leaching was started on April 1st. Rice was transplanted in the month of June in an area of 74 acres which gave an average yield of 14·8 maunds per acre. During Rab[illegible] area of 62·5 [illegible] maintained [illegible] Kharif 1941 was Rs. 727. The total produce of the landlord's share was 540 maunds of rice which was sold at Rs. 3-1-3 per maund, giving a total income of Rs. 1,660-13-0. The net income to the landlord is, therefore, Rs. 933-13-0.

(b) Lower Chenab Canal

Thatta Asalat—This village belongs to Dr. Sheikh Muhammad Alam, M.L.A., Barrister-at-Law. A considerable area of this village has deteriorated and has reached the Rakkar stage. As the village is situated at the tail of a minor only half a cusec of extra water-supply was given for reclamation.

Twenty-seven acres of rice were sown which gave an average yield of 12·1 maunds per acre.

During rabi 13·7 acres have been put under berseem. A summary of the accounts is given below :—

EXPENDITURE

——	Tenants' share	Landlord's share
	Rs. A. P.	Rs. A. P.
Land Revenue	27 0 0	27 0 0
Water Rates	110 0 0	110 0 0
Seed	11 4 0	11 4 0
Miscellaneous Expenses ..	..	25 0 0
Total ..	148 4 0	173 4 0

N. B.—Charges for transplanting, [illegible]

INCOME

	Rs.	A.	P.
Sale of 704·9 maunds of Jhona @ Rs. 3-5-8 ..	2,361	5	9
Sale of 125·4 maunds of Sathra @ Rs. 2-10-9 ..	335	0	0
Total income ..	2,699	5	9
Net income	1,289	3	0
Net income per acre	27	11	0

During rabi, 12 acres of gram were sown in wadh-wattar of rice.

Village Dubbar—This village belongs to the same type of landlords as that of village Jandraka, but the zemindars in this case took more interest and put 47 acres of rice on an extra supply of one cusec. To start with, the zemindars experienced considerable difficulty in constructing the watercourse in other men's land who had agreed to give land before the application was submitted but refused later. With the approval of the Superintending Engineer, the R. D. of the special outlet was changed to a site where zemindars could dig a watercourse in their own land. This made the sowing of rice late. The varieties Jhona and Sathra were sown which gave 15·2 and 8·9 maunds of yield per acre, respectively.

Harvesting of rice was also late which did not enable the zemindars to put the whole area under gram. Only 13·25 acres of gram were sown. Unlike the zemindars of Jandraka the landlords of Dubbar incurred practically the whole of the expenditure themselves and shared the produce with the tenants half and half. The costing account is given below :—

EXPENDITURE

	Tenants' share	Landlords' share
	Rs. A. P.	Rs. A. P.
Seed	..	25 14 6
Manure	..	..
Transplanting	89 0 0	..
Pay of Village Assistant ..	..	77 4 3
Land Revenue ..	..	139 4 0
Water-rates	..	691 0 0
Miscellaneous expenditure ..	..	52 9 0
Total ..	89 0 0	985 15 9

INCOME

	Rs. A. P.		Rs. A. P.
1. Sale of Sathra 41 maunds 31 seers at Rs. 2-10-0 per maund..	108 13 9	Sale of Sathra 41 maunds 31 seers at Rs. 2-10-0 per maund ..	108 13 9
2. Sale of Jhona 253 maunds 14 seers at Rs. 3 per maund ..	760 0 9	Sale of Jhona 253 maunds 14 seers at Rs. 3 per maund ..	760 0 9
Total ..	868 14 6	Total ..	868 14 6

STATEMENT SHOWING PROFIT OR LOSS PER ACRE

Tenants	Rs. A. P.	*Landlords.*	Rs. A. P.
Total expenditure ..	89 0 0	Total Eexpenditure ..	985 15 9
Total Income ..	868 14 6	Total Income ..	868 14 6
Net Income ..	779 14 6	Net loss ..	117 1 3
Total area under rice	47 acres	Area sown ..	47 acres
Profit per acre ..	Rs. 16·6	Net loss per acre ..	Rs. 2·5

Samra—In this village several zemindars joined to take a contract water-supply for reclaiming their thur fields. They worked in co-operation with each other in all reclamation operations including short-period waris. 1·5 cusecs of extra water-supply was allotted. The following varieties of rice were sown :—

—	Area sown in acres	Average yield per acre in maunds
Jhona	10·5	22
Sathra	61	27

A statement of accounts is given below :—

Kharif 1941

EXPENDITURE

	Rs. A. P.
1. Cost of seed	38 13 3
2. Transplanting charges	143 0 0
3. Water-rates	1,035 0 0
4. Land Revenue	183 12 0
5. Market expenses	139 9 6
6. Harvesting, threshing, cartage, etc. ..	658 2 9
7. Miscellaneous	100 0 0
Total	2,312 5 6

INCOME

Sale of 1,511 maunds of Jhona @ Rs. 3-1-7½	4,962	8	0
Sale of 220·6 maunds of Sathra @ Rs. 2-5-3	512	9	6
Total income	5,475	1	6
Total expenditure	2,312	5	6
Net income	3,162	12	0
Net income per acre	Rs. 11·2		

Chak 4-*R. B.*—Chak 4-R.B. is one village of an estate of 5,347 acres belonging to Sardar Sohan Singh, Rais, of Rawalpindi. In the old settlement records this land was classified first class. Signs of thur were noticed 15 years ago. In the year 1910 the landlord made a survey and discovered that out of the total area of 5,347 acres, approximately 1,500 acres were affected by thur. A large portion of this had completely gone out of cultivation. He applied for land reclamation in the year 1940, but the application was received late. In the year 1941 four cusecs of extra water-supply were sanctioned. As rapid deterioration was in progress the landlord thought that sooner or later almost the entire estate will become thur. He applied for permission for sowing rice in compact blocks irrespective of the fact whether a field was completely thur or in a stage of early deterioration. A soil survey was undertaken and it was found that in the compact blocks proposed the zone of salt accumulation was either at the surface or within 3 or 4 feet from it.

Reclamation was undertaken in two blocks; block A and block B. After leaching the areas given below were sown :—

Variety	Block A		Block B		Total	
	Area under rice	Yield per acre of rice	Area under rice	Yield per acre of rice	Area under rice	Average yield per acre
	Acres	Mds.	Acres	Mds.	Acres	Mds.
Sathra ..	18·5	29·1	6·3	18·3	24·8	23·7
Jhona ..	49·5	21 03	83·8	12·27	133·3	16·65
Mehlar ..	24·25	25·4	5·5	25·3	29·75	25·34

The combined average yield is 21·9 maunds per acre. The relatively low yield of rice in block B is due to the following reasons :—

(*i*) The land had been abandoned for years. The deterioration had reached the rakkar stage.

(ii) Most of the tenants had to come from long distances and were not familiar with the cultivation of rice.

During rabi the following crops have been sown :—

Block No.	Crop	Area in acres
A	Berseem ..	5·1
B	Berseem ..	7·4
A	Gram in wadh-watter	74·8
B	Gram in wadh-watter	12·8

The condition of the standing rabi crops is good and yields above normal are expected.

Costing accounts have been maintained. These are given below—

EXPENDITURE.

Tenants' share	Rs.	A.	P.	*Landlord's share*	Rs.	A.	P.
Cost of seed ..	104	7	3	Irrigation ..	30	13	0
Cost of manure ..	90	0	0	Pay of Village Assistant	120	0	0
Transplanting ..	331	1	0	Land Revenue ..	646	8	0
Land Revenue ..	646	8	0	Water Rates ..	1,290	0	0
Water Rates ..	1,290	0	0	Miscellaneous expenditure ..	218	9	0
Total ..	2,165	3	3	Total ..	2,305	14	0

INCOME.

Landlord's share

	Rs.	A.	P.
(i) Sale of Jhona and Mahlar 1,126 maunds 32 seers at Rs. 2-15-0 per maund ..	3,262	12	6
(ii) Sale of Sathra 268 maunds 20 seers at Rs. 2-6-3 per maund	642	7	0
Total ..	3,905	3	6
Net profit to the landlord	1,599	5	6
Profit per acre on area cultivated ..	[illegible]		

INCOME

Tenants' share	Rs. A. P.	Landlord's share	Rs. A. P.
1. Sale of Jhona and Mehlar (825 maunds 30 seers *plus* 316 maunds 20 seers) 1,142 maunds 10 seers at Rs. 3 per maund	3,426 12 0	Sale of Jhona and Mehlar (274 maunds 30 seers *plus* 103 maunds 20 seers) 378 maunds 10 seers at Rs. 3 per maund ..	1,134 12 0
2. Sale of Sathra 285 maunds 19 seers at Rs. 2-10-0 per maund ..	749 6 0	Sale of Sathra 100 maunds 13 seers at Rs. 2-10-0 per maund ..	263 5 6
Total ..	4,176 2 0	Total ..	1,398 1 6

PROFIT

Tenants	Rs. A. P.	Landlords	Rs. A. P.
1. Total expenditure ..	1,970 0 6	Total expenditure ..	*Nil*
2. Total income .	4,176 2 0	Total income ..	1,398 1 6
3. Net income ..	2,206 1 6	Net income ..	1,398 1 6
4. Area selected for reclamation .	100 acres	Area selected for reclamation ..	100 acres
5. Profit per acre ..	Rs.22 0 0	Profit per acre ..	Rs.14 0 0

This shows that the landlords obtained Rs. 1,408 from land which was previously thur without any investment on their part. This is equivalent to Rs. 14 per acre on the total area selected and Rs. 18 per acre on the area sown. The tenants incurred an expenditure of Rs. 1,970 and obtained a gross income of Rs. 4,176 which left them a balance of Rs. 2,206, i.e., Rs. 22 per acre on area selected and Rs. 27 per acre on area sown.

Village Janduwali—This village belongs to Dewan Brij Lal of Eminabad, the estate being under the management of the Court of Wards, Gujranwala. The Deputy Commissioner, Gujranwala, having received reports of the appearance of thur in this village applied on behalf of the Court of Wards for reclamation on extra supply of water. A soil survey was undertaken and it was found that although salt was a characteristic of the area it had appeared at the surface only in a portion of the estate. These fields had been abandoned for cultivation. In others patches of thur had started appearing. A compact

block of 50 acres was selected. Rice was sown in kharif 1941. The following are the results :—

—	Area sown in acres	Average yield per acre in maunds
Jhona—349	34·6	26·6
Sathra—278	15·3	24·0

During rabi gram has been sown in 47·5 acres in wadh-wattar of rice. The condition of the crop is satisfactory. A summary of accounts for kharif 1941 is given below :—

EXPENDITURE

		Rs.	A.	P.
1.	Cost of seed	25	15	6
2.	Transplanting charges	100	0	0
3.	Water Rates	648	0	0
4.	Land Revenue	148	9	0
5.	Harvesting, threshing and cartage	377	15	6
6.	Market expenses	118	5	6
7.	Miscellaneous	100	14	0
	Total ..	1,519	11	6

INCOME

		Rs.	A.	P.
1.	Sale of 917·7 maunds of Jhona @ Rs. 3-1-6	2,839	2	3
2.	Sale of 367·5 maunds of Sathra @ Rs. 2-15-1	1,081	8	0
	Total income .	3,920	10	3

Total produce 1,285 maunds 5½ seers.

	Rs.	A.	P.
Total expenditure ..	1,519	11	6
Net income .	2,380	14	9
Landlord's share	1,251	2	3
Tenants' share	1,129	12	6

The net income after deducting all expenses was divided half and half between the landlord and the tenants. The landlord in addition charged from tenants a sum of Rs. 56-1-3 as malikana.

Qila Ram Rang—This village also belongs to Dewan Brij Lal and is under the management of the Court of Wards, Gujranwala. In this estate approximately 300 acres have gone out of cultivation. The results of the soil survey showed that at least three rice crops would be required for complete reclamation. In the year 1941 one cusec of extra water-supply was allotted. The following varieties of rice were sown :—

—	Area sown in acres	Average yield per acre in maunds
Jhona	40·4	21
Sathra	11·6	17·8

During rabi 1941-42 gram has been sown in 14·25 acres and berseem in half an acre. The decrease in area under rabi crops is due to the fact that the zemindars were late in taking the extra water-supply. This made the rice crop late. As rice was harvested late gram could not be sown in the whole of the area.

A summary of the accounts is given below :—

EXPENDITURE

		Rs.	A.	P.
1.	Cost of seed	25	14	6
2.	Transplanting charges	104	0	0
3.	Water Rate	690	0	0
4.	Land Revenue	137	6	0
5.	Harvesting, threshing, cartage, etc. ..	363	12	6
6.	Market expenses	157	14	0
7.	Miscellaneous	88	14	0
	Total ..	1,567	13	0

INCOME

	Rs.	A.	P.
839·25 maunds of Jhona at Rs. 3-9-0 ..	2,989	13	3
204·1 maunds Sathra at Rs. 2-6-6	490	14	0
Total income ..	3,480	11	3
Total expenditure ..	1,567	13	0
Net income ..	1,914	11	0

	Rs.	A.	P.		Rs.	A.	P.
Landlord's share ..	957	5	6	Tenants' share ..	957	5	6

Bacha Kohna—The whole of this village belongs to Dewan Iqbal Nath of Eminabad. A survey of the village was undertaken which showed that fields in all stages of deterioration existed. There were fields which had become completely thur and had been lying uncultivated for several years. In others although crops were sown the yields were low. A portion of the remaining area was carrying normal crops while in another portion patches of salt were present. Two cusecs of extra water-supply were sanctioned for the reclamation of thur land in this village. In the first year the completely deteriorated land was handled. The following are the areas sown :—

	Area sown in acres	Average yield per acre in maunds
Jhona	64·5	20·3
Sathra	16·7	12·7

A summary of the accounts is given below :—

KHARIF 1941

Expenditure

		Rs.	A.	P.
1.	Cost of seed	51	4	3
2.	Transplanting charges	108	0	0
3.	Water-rates	1,296	0	0
4.	Land Revenue	257	9	0
5.	Labour employed (for direct cultivation) ..	288	0	0
6.	Harvesting, threshing, cartage, etc. ..	492	0	0
7.	Market expenses	224	11	3
8.	Miscellaneous	227	0	0
	Total	2,944	8	6

INCOME

	Rs.	A.	P.
Sale of 1,364 maunds 4 seers of Jhona @ Rs. 3-7-0 per maund . .	4,689	2	3
Sale of 238 maunds 1 seer of Sathra @ Rs. 3-6-6 per maund	810	12	3
Total income .	5,499	14	6
Total expenditure ..	2,944	8	6
Net income ..	2,555	6	0
Net income per acre	31	7	6
Landlord's share	1,327	1	6
Tenant's share	1,228	4	6

During rabi the following crops have been sown :—

	Acres
Gram	31·8
Berseem	31·1

The condition of the rabi crops is good.

Bacha Nau—This village belongs to the Gurdwara Parbandhak Committee, Bacha Nau. A large portion of this village has become thur and is lying uncultivated for years. On an application from the Gurdwara Parbandhak Committee two cusecs of extra water-supply were sanctioned by the Land Reclamation Board. The following are the areas sown :—

—	Area sown in acres	Average yield per acre in maunds
Jhona	65·9	26
Sathra	21·7	16

The Gurdwara Committee decided to give the whole of the land to the tenants who were asked to bear all expenses of reclamation and give a nominal share to the landlord out of the produce. From the accounts maintained the total expenditure incurred was Rs. 2,971-2-0. The total income obtained was Rs. Rs. 6,975. Deducting expenditure net income was Rs. 4,003-14-0, This was divided between the Gurdwara Committee and the tenants, the Gurdwara Committee receiving Rs. 430-3-0 and the tenants Rs. 3,573-11-0.

Kot Dewan Ali—An account of reclamation for the year 1940 was given in the last Annual Report. One cusec of additional water-supply was sanctioned for reclamation. Pir Dewan Ali Shah, the proprietor of this village, applied that his zemindari supply should also be converted into a contract supply so that he is able to devote the whole of the water towards reclamation. This was sanctioned by the Chief Engineer. During Kharif 1941, 123 acres of rice were sown on a total supply of 2·5 cusecs. The Pir Sahib did not maintain accurate costing accounts but from the figures supplied by him, his total expenditure was Rs. 2,226. His share of rice was approximately 1,500 maunds which was sold for Rs. 5,000. This gave a profit of Rs. 2,774 to the Pir Sahib. A portion of this village has now carried two rice crops and it is considered that some fields are fit for normal cropping. A soil survey is in progress. During Rabi 1941-42, 28 acres of gram and berseem have been sown and 48 acres have been put under wheat after an application of farm-yard manure

Chak 589-*G.B.*—This was the first year of reclamation in Chak 589-G.B. in land belonging to Messrs. Baij Nath and Jagan Nath Bhalla and Malik Mohammad Hayat Khan. The extra water-supply sanctioned for reclamation was 0·75 cusec.. It was added in the existing water-course and the warabandi was changed by the Executive Engineer. As the other share-holders did not agree to a five-day warabandi, a 7-day wari was sanctioned. For the area managed by Messrs. Jagan Nath and Baij Nath Bhalla, they have carried out the experiment according to instructions and have submitted a report which is reproduced below—

"We undertook reclamation of our Thur land, situated on the outlet at R. D. 55,931-R, Nankana Minor, Lower Chenab Canal on ·5 cusec of contract supply of water, which was given to us for kharif 1941.

We took very great pains in supervising every minute detail of the various operations connected therewith and made every possible effort to make the experiment a success. We are glad to submit herewith the figures for your perusal, which speak for themselves. There is no doubt that the price of rice crop sold was favourable, which also contributed to the success we have been able to achieve in our net results.

The seeds of rice (Sathra 278 and Jhona white No. 349) were sown on 17th May 1941 and a little on the 28th May, 1941, to supplement any shortage that might arise. We started transplanting on 21st June and completed the same about the middle of July, the total area transplanted being 21½ acres.

The whole rice crop has since been cut and sold, the income and expenses incurred thereon up to the end of November 1941 are as follows :—

Income—Price received for sale of 185 maunds 24 seers 12 chhataks of rice being our share—Rs. 583-4-3 only.

Expenses—Amount paid to canal department as cost of—

	Rs.	A.	P.
0·5 cusec of water	324	0	0
Cost of manure purchased for sowing of rice paniri	9	0	0
Cost of rice seed	13	1	3
Miscellaneous	95	8	0
Land Revenue	100	14	3
Cost of transplantation	51	3	0
Contribution towards the pay of the Village Assistant	28	0	0
Less amount realised from tenants, as per details given below	145	8	9
Net expenses ..	476	1	9

Costing accounts will be given in the next year's report.

Chak 236-G. B. (belonging to Raja Sir Daya Kishen Kaul)—In kharif 1940 the Raja Sahib was late in applying for extra water-supply. This made the rice sowing late. Leaching was also insufficient. Only 44 acres of rice were sown which gave an average yield of 18 maunds per acre. During Rabi 1940-41 the following crops were sown :—

	Acres
Gram	26·6
Berseem	14·6

Gram gave an average yield of 16 maunds per acre and berseem fetched the normal price.

From the results of analyses after rice and the yields of gram and berseem some fields were declared as reclaimed. Cotton was sown in 3·7 acres. This gave an average yield of 8·4 maunds per acre.

During kharif 1941, 76 acres of rice were sown which gave an average yield of 15 maunds per acre.

During Rabi 1941-42 the following crops have been sown :—

	Acres
Berseem	31
Gram in wadh-watter	55

The condition of the rabi crops is good.

With one year's experience of reclamation and the yields of rice and rabi crop of gram and berseem and also the rate of damage due to thur the Raja Sahib applied for the water-supply to be increased so that he could finish reclamation in a shorter period. For kharif 1942 the extra water-supply has been raised from two to four cusecs. This will enable the Raja Sahib to put the whole of the existing thur area under leaching and rice.

(c) Lower Jhelum Canal

In the Lower Jhelum Colony reclamation on contract supplies of water was continued in villages Sher Muhammadwala and Fatehpur.

In 1941 one cusec of extra water was sanctioned for the zemindars of chak 10-N.B. The new five-day warabandi for this outlet could not be sanctioned before May 15th. This made leaching late and the rice transplanting was also done late. Rice was sown in an area of 35 acres out of which 3 acres was a failure. The total produce obtained by the zemindars was 242 maunds which gave an average

yield of 8 maunds per acre. Gram has been sown in wadh-wattar of rice. Accounts for kharif 1941 are given below :—

EXPENDITURE

	Rs.	A.	P.
Cost of 1.0 cusec water-supply	498	3	0
Miscellaneous expenses	120	0	0
Cost of $7\frac{1}{2}$ maunds of rice seed	26	15	0
Land revenue of 35 acres and 10 marlas at Rs. 3-8-0 per acre	122	11	6
Total ..	767	13	6

INCOME

Total area of rice sown		35 acres
	Md. Sr.	
Total out turn of rice	242 11	
Wages in kind	12 4	
Net yield recovered	230 7	
	Rs. A. P.	
Income of the zemindars from the rice crop sold at Rs. 2-12-0 per maund .	632 15 9	

N.B.—Land owners themselves cultivate the land.

In spite of the fact that the zemindars have incurred a loss of Rs. 4 per acre on rice they are satisfied with the improvement of the land. After the harvesting of the wadh-wattar crop of gram the zemindars will certainly get a profit. For kharif 1942 the zemindars of this village and also of the adjoining villages have applied for extension of reclamation on contract supplies of water.

One difficulty in making reclamation successful in the Lower Jhelum Canal area is the closures during kharif. These closures adversely affect the process of leaching and the growth of rice. The yields of rice are low which reduce the zemindars' income.

Kharif closures on this canal were introduced to prevent rise in the water-table. The policy of using closures as an anti-water-logging measure has been abandoned on other canals. Taking into consideration the fact that thur is rapidly increasing in this colony and that reclamation will have to be taken up soon it is advisable that the kharif closures should be abandoned.

(d) UPPER CHENAB CANAL

Dahranwali This is another instance of several zemindars co-operating amongst themselves to undertake reclamation of thur land. At the request of the zemindars a soil survey was undertaken which showed that their thur land could be reclaimed with three rice crops. The area is irrigated from the Nokhar Branch which was

an application of farmyard manure. It is expected that gram will give an average yield of 8 maunds per acre. From the zemindar's test that where gram grows the land should be considered as reclaimed, the Raja Sahib is satisfied with the progress of reclamation.

A soil survey after rice is in progress. Even though gram is successful it is considered that the land is not completely reclaimed. It will probably require one more rice crop before it can be handed back to the landlord for cultivation on zemindari water-supply.

Costing accounts for kharif 1941 are given below :—

INCOME

Name of Variety	Total yield	Rate per maund	Value
	Mds. Srs. Chs.	Rs.	Rs. A. P.
Jhona	1,382 26 0	4	5,530 0 0
Sathra	188 0 0	4	752 0 0
Mehlar	545 30 0	4	2,183 0 0
Total ..	2,116 16 0	..	8,465 0 0

	Rs. A. P.
Landlord's share	4,232 12 9
Tenants' share	4,232 12 9

EXPENDITURE

—	Paid by landlord	Paid by Tenants
	Rs. A. P.	Rs. A. P.
1. Price of 2·5 cusecs of water	765 0 0	520 0 0
2. Land revenue ..	338 7 0	..
3. Seed	80 0 0	..
4. Transplanting charges at Rs. 2 per acre ..	..	216 0 0
5. Harvesting	157 4 9	157 4 9
6. Miscellaneous ..	113 0 0	..
Total ..	1,863 11 9	1,223 4 9

	Rs.	A.	P.
Landlords' income	4,232	12	9
Landlords' Expenditure	1,863	11	9
Profit	2,369	1	0
Tenant's income	4,232	12	9
Tenant's Expenditure	1,223	4	9
Profit	3,009	8	0

The Salt Zone in the soil profile and the Tirak attack on American cotton in the Montgomery District—The question of the tirak attack on American cotton in the Punjab was discussed in the 26th meeting of the Waterlogging Board held on 2nd December 1940. As a result of the discussion it was suggested that the reports of the district officers showed a state of affairs comparable to the picture of the salt zone in the soil profile and indicated that the appearance of tirak was a symptom of the approach of thur. This observation harmonised with the fact that in reclaimed areas where the salt zone had been depressed cotton was not attacked by tirak. It was decided that, while application of nitrogenous manures and late sowing should be accepted as a possible treatment, the recommendations of the Director of Irrigation Research regarding the removal of the salt zone from the soil profile for the prevention of the tirak attack should be further investigated.

It is well-known that the tirak attack is more severe in the Lower Bari and the Nili Bar Colonies and enquiries show that the tendency of the zemindars is to reduce the area under American cotton. A few examples are given below :—

Serial No	Name of Chak	American cotton sown during kharif 1940	American cotton sown during kharif 1941
		Acres	Acres
1	15/11-L	141	15
2	36/12-L	83	13
3	38/12-L	103	44
4	39/12 L	85	23

An estimate of the loss of Government revenue can be made from the fact that a sum of Rs. 45,000 was remitted on account of tirak attack during kharif 1940 in the Chichawatni Sub-Tahsil al

For the investigations proposed, Government sanctioned an anti-tirak centre to be established in the Montgomery District. A budget of Rs. 10,000 was sanctioned for this purpose. A survey of cotton fields in the Chichawatni area was carried out by the Executive Engineers and the local civil officers. Approximately 2,000 acres belonging to 600 zemindars were selected for treatment. A soil survey was undertaken by the Land Reclamation Department, and from the results of analyses it was found that in all the fields the salt content or the alkalinity was high at some depth of the profile within 6 feet from the natural surface.

Extra water on the basis of one cusec for 50 acres was added to the existing water-courses and the warabandis reduced to five-day waris. As the experiment does not relate to normal reclamation but to the depression of the zone of accumulation of salt, leaching was started on May 15th as extra water was not available before that date. Rice was transplanted in the month of June. The extra water-supply was withdrawn on the 1st of October. Gram and berseem have been sown in wadh-wattar of rice. The condition of gram after rice is excellent and is much better than that grown in the adjoining undeteriorated fields. Cotton will be sown in kharif 1942.

The staff provided for the supervision of rice cultivation in the anti-tirak centre took an opportunity of studying the accounts of the zemindars and the cost of the experiment to the Government. A summary of the accounts is given below :—

Total area under rice	1,721 acres
Total yield of rice	22,519 maunds
	Rs.
Total income to the zemindars	83,101
Income to the zemindars after deducting their expenses	55,603
Total budget sanctioned by Government ..	10,000
Water rates paid	9,153
Land Revenue paid	5,991
Total income to Government	15,147

The Chakanwali Experimental Reclamation Station—Practically the whole of the area neglected by the lessee during the period of the lease has been brought under cultivation again. During the year all this area produced normal crops of rice, sugarcane, wheat, berseem, and other fodders.

One hundred and thirty-one acres of new Rakkar land were broken and reclamation started. It is expected that this area will be partially reclaimed in 1942 and that the costs will be covered by the income from rice.

The sowing of gram in wadh-wattar of rice has been introduced in the tenants' area. During rabi 1940-41 it gave an average yield of 13·3 maunds per acre. This has increased the zemindars' income and Government revenue from the farm area. Previously the land used to lie fallow and rice always followed rice. In fields in which gram has been successful it is proposed to sow normal crops; the water-supply for rice will be utilised for the reclamation of the adjoining thur fields.

During kharif 1941 great difficulty was experienced in procuring rice seed for the Reclamation Centres. The total quantity required was not available with the Agricultural Department. The seed available at Chakanwali in Government godowns or with the tenants was all sold and in kharif 1941 new seed from Kala Shah Kaku was introduced. From this year's crop approximately 3,000 maunds of rice have been utilised by the Land Reclamation Department for distribution as seed.

During the year 1941-42 the total expenditure on the farm was Rs. 30,000 and the total expected income not less than Rs. 56,000.

SOIL AND WATER SURVEYS

(i) *Soil Survey of a Million acres in the Lyallpur District*

In the report for the year 1941 the results of a preliminary soil survey of the area irrigated by the Nasrana distributary of the Jhang Branch were given and it was stated that in certain villages as much as 23 per cent of the cultivated area had developed salts at the surface. On the basis of the results of this preliminary survey Government decided to have an area of a million acres checked to determine :—

(*a*) Areas where deterioration is in progress and also the rate of deterioration.

(*b*) the depth of the soil crust with a view to locate areas where the rising water-table is likely to cause waterlogging.

In October 1941, four soil survey parties were organised for the field work and two Research Assistants were deputed for the laboratory work. The whole of the survey is under the charge of an experienced officer.

During the period ending 31st March 1942 field work was finished in 137 villages comprising an area of 100,000 acres. The results of the survey are shown in Fig. 230. The following are the remarks :-

(*a*) *Depth of the Soil Crust*—The depth of the soil crust varies from place to place. In a large portion of the area so far surveyed field observations show that it is less than 10 feet. In some pits a phenomenon was observed which is different to the conception of the soil crust. In these pits a layer of sand varying from 1 foot to 2 feet

in thickness appeared at a depth of 5 to 6 feet from the natura surface below which loam or clay loam existed and sand appeared agai at a depth of 9 to 10 feet.

(b) *Texture of the Soil Profile*—Three types of profiles have bee observed during the course of the survey :—

(i) First 3 to 4 feet of the profile are loam or light loam Below this depth the sand content increases till at a depth of 7 f or 8 feet the sand is all coarse.

(ii) First 5 feet to 6 feet of soil is loam or light loam ; th there is a clay layer varying from 9 inches to 2 feet in thickn below which there is coarse sand.

(iii) First 3 feet to 4 feet of soil is sandy loam. Then 2 fe to 3 feet of loam which gradually changes into clay to a depth 10 feet.

(c) *Zone of accumulation of salt*—It was not possible to leave th soil profiles exposed for some time so that the zone of salt accumulation could be seen. It was observed however that where the zone o accumulation was at a depth of 0 to 4 feet the characteristic sal appearance was noticed in the course of a day or so after digging.

(d) *Kankar Layer*—Kankar is met with in almost all soils The depth and the size of kankar varies from place to place. Observations on the growth of crops with reference to the depth of the kankar layer, its size and colour and also the appearance of thur at the surface and alkalinity of the subsoil layers are in progress.

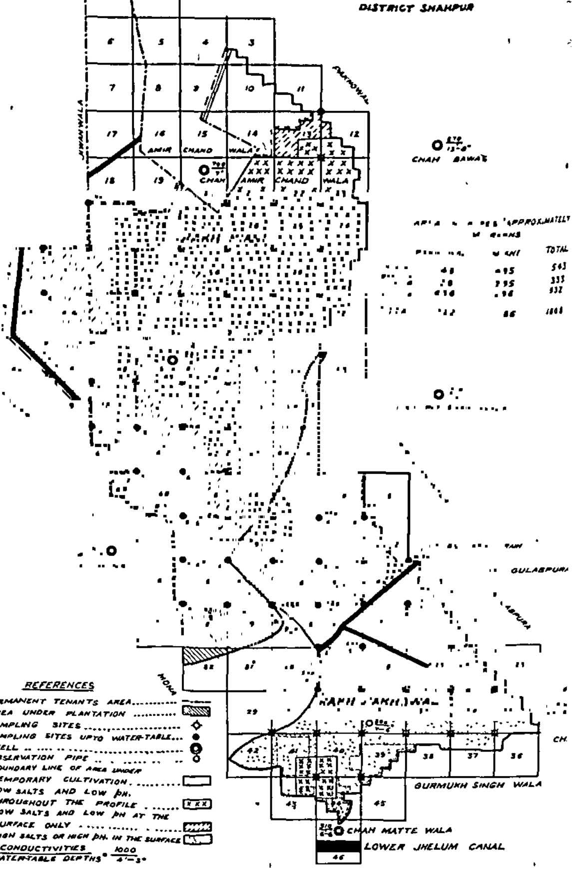
(e) *Spread of Thur*—From Fig. 230 it will be seen that there is hardly a village where thur is non-existent. From enquiries made it can be stated that the appearance of thur is on the increase and every year cultivated land goes out of cultivation due to this. In the south-eastern portion of the district thur girdawaris are carried out and land reclamation operations are also in progress. In the area irrigated by the Khurrianwala distributary, however, the water-table is at a depth of 30 feet or more from the natural surface and no thu girdawaris are undertaken. The soil survey has revealed that this area is under rapid deterioration and that if nothing is done to prevent this, the stage of uneconomic yields will soon be reached and zemindars will have no other alternative but to abandon cultivation.

(ii) Soil and Sub-soil Water Surveys of Miani and Pakhowa Rakhs in the Bhalwal Tahsil, District Shahpur.

Proposals were made by the Deputy Commissioner, Shahpus district, that the area, covered by Rakhs Miani and Pakhowal should be given on lease on conditions that the lessees should sink open wells for providing irrigation. Before giving approval to the proposal, the Financial Commissioner, Revenue asked for a soil and water survey to be undertaken to investigate if the areas were suitable for allotment and the subsoil waters fit for irrigation.

FIG. 231

SOIL MAP OF RAKHS PAKHOWAL AND NIANI
DISTRICT SHAHPUR



From the examination of the soil profiles in the field and soil samples in the laboratory the whole area of Rakhs Miani and Pakhowal has been divided into three types, Fig. 231.

In type 1 the soil profile has low salts and low pH in the whole of the profile to a depth of 6 feet. Normal yields will be obtained on this type of land if irrigation water is available.

In type 2 although the salt content and pH values of the surface soil are low they are high at some depth of the soil profile within 6 feet from the natural surface. This type of land is likely to deteriorate or give reduced yields after some time.

In type 3 the salt content or the pH value is high at the surface. This land requires reclamation with canal water before it can be allotted for cultivation.

The major portion of Rakh Pakhowal belongs to type 3 and has reached the Rakkar stage. It is highly alkaline and at places is very hard at the surface. This land is not economically reclaimable. Looking at the map it will be seen that the area from the Lower Jhelum Canal Main Line to as far as Pakhowal Railway Station is of this type. The water-table is very near the surface and a very intensive system of field drains would be necessary if an attempt is made to reclaim the area. Reclamation of this area on well irrigation is impossible.

The total area of the Rakhs and the areas under each type are given on the map, Fig. 231.

Conductivities of subsoil water samples taken from bores freshly dug are also given on the map. These conductivities are on the high side, but samples of water taken from wells in the adjoining area show that they are suitable for irrigation. The analyses of waters are given in Table LXVIII.

Waters whose "Salt Index" is negative are suitable for irrigation. Those which have a positive "Salt Index" are unfit for irrigation. The only water unsuitable for irrigation is from Chah Pir Shahwala, Regd. No. 37.

From the analyses of the soils and the well waters the following are the conclusions :—

During the period 15th June 1941 to 1st October 1941 the run-off from the slopes of the outer hills was received only once in some of the wahns and twice in others. The sites of sampling for run-off water from the outer hills are marked 4, 5, 6, 7 and 8. They relate to the following wahns :—

Site No. 4	.. ..	Kohalian Wahn
„ „ 5	.. ..	Kachoola „
„ „ 6	.. ..	Dhaddar „
„ „ 7	.. ..	Dangali „
„ „ 8	.. .. ..	Trimma „

The results of the analyses are given in Table LXXII. Compared with the salt content of the waters in the main Nullah the salt content of the waters from the slopes of the outer hills is low although the total salt content is above the limit of 60 parts per 100,000 which is the limit for good water for irrigation. The main salt present in these waters is calcium sulphate which is harmless. It is considered that these waters would not damage the land. The water that will be harmful is the run-off water from Trimma Wahn. If this wahn is trained so as to join the neighbouring stream the addition of salt to the fields flooded by this wahn can be prevented.

During the course of the survey, the Research Assistants made observations and enquiries regarding the spread of the saline flood-waters on the land between the hills and the railway line, as distinct from the spills from the Makrach Nullah.

Their observations showed that water from Kohalian Wahn does not spread over a very large area as this joins the Makrach Nullah immediately below the point where it comes from the hills. The water from the Kachoola Wahn is not very saline. This is utilised by zemindars for irrigating their fields by means of kuhls (water-courses) which are taken off from the wahn by constructing small earthen bunds. Water that is not utilised by the zemindars goes into the Golpur Branch. Although kuhls have also been taken off the remaining three wahns, viz., Dhaddar, Dangali and Trimma, most of the water spreads on the land between the railway line and the hills and is all absorbed.

TABLE LXIV—CONTINUED

Name of village	Harvest	Total C. C. A.	Total area Thur	Percent Thur on C. C. A.
		Acres	Acres	
Machhrala—concld ..	1936 Kharif	8,538	674	7·9
	1937 Rabi	8,537	887	10·4
	1937 Kharif	8,537	927	10·9
	1938 Rabi	8,537	1,137	13·3
	1938 Kharif	8,537	1,231	14·4
	1939 Rabi	8,537	1,193	14·0
	1939 Kharif	8,537	1,156	13·5
	1940 Rabi	8,537	1,014	11·9
	1940 Kharif	8,532	1,116	13·1
Kot Shah Mohd. ..	1935 Rabi	2,569	594	23·1
	1935 Kharif	2,562	788	30·8
	1936 Rabi	2,562	834	32·6
	1936 Kharif	2,562	857	33·4
	1937 Rabi	2,562	759	29·6
	1937 Kharif	2,567	938	36·5
	1938 Rabi	2,567	957	37·2
	1938 Kharif	2,567	909	35·4
	1939 Rabi	2,567	841	32·8
	1939 Kharif	2,567	803	31·3
	1940 Rabi	2,568	529	20·6
	1940 Kharif	2,568	744	29·0
575-G. B. ..	1935 Rabi	1,710	89	5·3
	1935 Kharif	1,710	213	12·4
	1936 Rabi	1,710	262	15·3
	1936 Kharif	1,587	316	19·9
	1937 Rabi	1,586	320	20·2
	1937 Kharif	1,587	349	21·9
	1938 Rabi	1,587	379	23·83

TABLE LXIV—CONTINUED

Name of village	Harvest	Total C. C. A.	Total area Thur	Percent Thur on C. C. A.
		Acres	Acres	
575-G. B.—*concld.*	1938 Kharif	1,587	377	23·7
	1939 Rabi	1,587	369	23·2
	1939 Kharif	1,587	403	25·4
	1940 Rabi	1,587	401	25·2
	1940 Kharif	1,587	383	24·1
Bagharai ..	1935 Rabi	2,351	112	4·0
	1935 Kharif	2,373	187	7·8
	1936 Rabi	2,373	210	8·8
	1936 Kharif	2,373	337	14·2
	1937 Rabi	2,373	396	16·7
	1937 Kharif	2,373	484	20·3
	1938 Rabi	2,373	544	22·9
	1938 Kharif	2,375	547	23·0
	1939 Rabi	2,375	561	23·6
	1939 Kharif	2,375	528	22·2
	1940 Rabi	2,375	516	21·7
	1940 Kharif	2,375	530	22·3
Sharinjhangar ..	1934-35	11,561	..	..
	1936 Rabi	11,561	576	[illegible]
	1936 Kharif	11,633	874	7·5
	1937 Rabi	11,633	[illegible]	[illegible]
	1937 Kharif	11,643	1,162	[illegible]
	1938 Rabi	11,643	[illegible]	11·7
	1938 Kharif	11,643	[illegible]	[illegible]
	1939 Rabi	11,642	[illegible]	[illegible]
	1939 Kharif	11,642	1,941	16·6
	1940 Rabi	11,642	1,922	16·5
	1940 Kharif	[illegible]	[illegible]	16·5

TABLE LXV

STATEMENT SHOWING AN ABSTRACT OF COSTING ACCOUNT OF NANKANA SAHIB RECLAMATION CENTRE FOR KHARIF 1941

Chak No.		Area sown, in acres	Area failed, in acres	Area harvested, in acres	Total expenditure including land revenue and water rate	Total outturn: Grain	Total outturn: Cash	Balance with zamindar	Net profit per acre
					Rs.	Maunds	Rs.	Rs.	Rs.
Feroze	..	83·4	11·6	71·8	650·0	667·5	2,336·0	1,686·0	23·4
Chak 572	..	223·2	12·6	210·6	2,274·0	2,521·8	8,822·0	6,548·0	31·1
Machhrala	..	320·9	5·1	315·8	4,015·0	4,823·1	16,058·0	12,044·0	38·1
Parianwala	..	134·1	0·3	133·8	1,506·0	2,167·3	6,502·0	4,996·0	37·3
Chak 559	..	146·4	2·9	143·5	1,226·0	1,611·0	6,574·0	5,347·0	37·8
Chak 560	..	75·9	7·1	68·8	721·0	734·0	2,569·0	1,848·0	26·8
Chak 562	..	89·9	2·2	87·7	770·0	849·4	3,657·0	2,887·0	32·9
Chak 563	..	40·6	0·9	39·7	389·0	351·2	1,229·0	840·0	21·1
Chak 567	..	21·0	1·5	19·5	186·0	236·8	947·0	761·0	39·0
Chak 586	..	20·0	..	20·0	176·0	264·7	1,024·0	849·0	42·4
Chak 588	..	44·6	..	44·6	613·0	568·0	1,988·0	1,375·0	30·8
Chak 371	..	54·1	0·3	53·8	276·0	532·3	1,597·0	1,321·0	24·5
Lundianwala	..	48·9	0·3	48·6	604·0	508·3	1,770·0	1,176·0	24·1
Chak 658	..	14·3	..	14·3	185·0	189·0	756·0	570·0	40·0
Total	..	1,317·3	44·8	1,272·5	13,591·0	16,025·3	55,838·0	42,247·0	..

Net profit per acre for the centre .. Rs. 32·1

TABLE LXVI

STATEMENT SHOWING AN ABSTRACT OF COSTING ACCOUNTS OF DANGALI RECLAMATION CENTRE

Serial No.	Chak No	Area sown	Area failed	Area harvested	Total expenditure excluding land revenue and water-rate	Total outturn: Grain	Total outturn: Cash	Balance with Zemindars	Profit per acre
		Acres	Acres	Acres	Rs.	Maunds.	Rs.	Rs.	
1	626-G. B.	107·1	1·3	105·8	669·0	1,070·8	4,283·0	3,614·0	33·8
2	627-G. B.	114·3	2·0	112·3	737·0	1,528·6	6,035·0	5,298·0	46·4
3	628-G. B.	112·7	2·6	110·1	568·0	852·8	3,411·0	2,843·0	25·2
4	629-G. B.	174·6	3·5	171·1	669·0	1,889·6	7,559·0	6,890·0	39·4
5	631-G B.	29·8	..	29·8	114·0	48·7	195·0	81·0	2·7
6	233-G. B.	64·2	..	64·2	392·0	638·1	2,553·0	2,161·0	33·6
7	234 G. B.	12·2	..	12·2	95·0	158·3	633·0	538·0	44·1
8	235-G. B.	69·0	.	69·0	496·0	898·0	3,592·0	3,096·0	44·8
9	237-G. B.	215·3	5·2	210·1	479·0	1,391·3	5,560·0	5,081·0	23·6
10	648-G. B.	72·0	0·8	71·2	472·0	557·3	2,229·0	1,757·0	24·4
11	144-G. B.	90·7		90·7	625·0	1,075·6	4,302·0	3,677·0	40·5
12	19-G. B	94·9	3·3	91·6	612·0	926·0	3,704·0	3,092·0	32·6
13	11-G. B.	36·7	..	36·7	279·0	627·8	2,511·0	2,232·0	60·8
14	20-G. B.	45·9		45·9	289·0	667·0	2,668·0	2,378·0	51·7
15	124-G. B.	136·2		136·2	686·0	1,882·3	7,529·0	6,843·0	50·2
16	125-G. B.	173·2	6·7	166·5	981·0	[illegible]	5,706·0	4,725·0	27·2
17	126-G. B.	76·7		76·7	577·0	770·8	[illegible]	2,734·0	[illegible]
18	[illegible] G. B.	44·2		44·2	212·0	[illegible]	[illegible]	915·0	21·3
19	61 G. B.	102·4	0·6	101·8	691·0	923·2	[illegible]	[illegible]	29·2
	Total ..	1,772·1	26·0	1,746·1	[illegible]	17,627·9	[illegible]	[illegible]	

Rs.

Profit (excluding Land Revenue and Water Rate) per acre for the Centre ... [illegible]

Balance with Zemindars ... [illegible]

Land Revenue and Water Rate ... [illegible]

[illegible]

[illegible]

TABLE LXVII

RECLAMATION ACCOUNT OF BLOCKS A AND B IN RENALA ESTATE

Year	Estates' share only	Expenses	Income
1940		Rs. A. P.	Rs. A. P.
November ..	By profit brought forward from last year.	..	6,532 15 3
Ditto ..	To Kuppas overestimated last year ..	732 12 0	..
Ditto ..	To Goor and Shukkar ..	399 3 0	..
Ditto ..	By additional c/o Moonji short estimated last year.	..	20 0 0
Ditto ..	By Harhar 6 maunds at Rs. 2 ..	..	12 0 0
Ditto ..	By Rice straw sold ..	..	4 1 0
Ditto ..	To cartage of manure to Reclamation area	81 0 0	..
November 11th ..	To c/o 2 Iron Pans in October ..	36 12 0	..
November 12th ..	To 76 culture from Lyallpur ..	39 8 0	..
November 25th ..	To wages to Beldars for October ..	29 6 0	..
December 10th ..	To 2 filters purchased for reclamation area	15 0 0	..
December 18th ..	To wages to beldars and Munshi for November.	32 4 0	..
December 20th ..	To stationery in November ..	0 3 3	..
December 22nd ..	To labour and stores on works in November.	33 5 9	..
December 24th ..	To Railway freight on culture from Lyallpur.	2 7 0	..
December 26th ..	To Travelling Allowance to a clerk for Lahore trip on reclamation.	7 10 0	..
1941			
January 4th ..	To stores in December ..	2 3 6	..
January 18th ..	To labour and stores on works in December.	26 7 3	..
January 26th ..	By sale of green gram ..	..	39 5 9
Ditto ..	To c/4 copies Research Publication of Director of Irrigation Research.	5 2 0	..
Ditto ..	To labour engaged in December ..	21 8 0	..
	Carried over ..	1,484 11 9	6,608 6 0

TABLE LXVII—CONTINUED.

Year	Estates' share only	Expenses	Income
1941—*contd.*		Rs. A. P.	Rs. A. P.
	Brought forward ..	1,484 11 9	6,608 6 0
January 31st ..	To hire of estate car, October to December.	3 6 0	..
February 8th ..	To stationery in January ..	0 14 0	..
Feb uary 14th .	To stores—Ammonium Sulphate supplied in January.	330 2 6	..
February 20th .	To hire of estate car, 7 miles in January at 0-1-9 per mile	0 12 3	..
February 26th ..	To salary to Khushi Mohammad, Ilaqadar for January.	20 0 0	..
March 4th ..	To stationery in February ..	0 2 9	..
March 10th .	To repi to blacksmith and carpenter for Kh 40.	04 0 0	.
March 22nd ..	To hire of a car 11 miles, February ..	1 6 0	..
March 15th ..	To salary to Ilaqadar for February ..	20 0 0	..
March 25th .	By cash sale of green berseem ..	..	77 12 0
March 31st ..	By cash sale green berseem .	..	467 8 0

TABLE LXVII—continued.

Year	Estate's share only	Expenses	Income
1941—*contd.*		Rs. A. P.	Rs. A. P.
	Brought forward ..	2,385 6 9	7,309 14 0
May 6th ..	To stationery and stores in April ..	16 3 6	..
May 13th ..	By sale of green berseem ..	..	5 2 0
June 7th ..	To salary to Ilaqadar for May ..	20 0 0	..
June 15th ..	To stores—Sulphate of ammonia supplied in May.	104 11 9	..
June 26th ..	To wages for cutting berseem ..	8 8 0	..
June 28th ..	To labour and stores on works in May	3 9 6	..
July ..	To stores—ammonium sulphate lantern globes, etc., in June.	139 2 6	..
July 2nd ..	To dry cells for torch and bulbs ..	1 2 6	..
Ditto ..	To stationery in June ..	1 9 6	..
July 4th .	To salary of Ilaqadar for June ..	20 0 0	..
July 15th ..	To labour and stores on works in June	1 11 6	..
July 26th ..	To berseem seed damaged .	2 0 0	..
August 3rd ..	To stationery and stores in July ..	19 9 0	..
Ditto ..	By cash sale of rice nursery .	..	2 0 0
Ditto ..	To salary to Ilaqadar for July ..	20 0 0	..
August 20th ..	To labour and stores on works in July	41 3 0	..
September 3rd .	To stationery supplied during August	6 10 9	..
September 14th ..	To share c/o berseem culture ..	67 10 0	.
Ditto ..	To salary to Ilaqadar for August ..	20 0 0	..
September 27th ..	To Railway freight on culture from Lyallpur.	1 12 0	..
September 30th ..	To additional c/o berseem culture ..	0 14 0	..
Ditto ..	To Railway freight on 1 case culture from Lyallpur.	1 10 0	..
Ditto ..	To labour on clearing berseem seed ..	7 12 0	..
Ditto ..	To seeds supplied—Munji and berseem—during ½ year April to September 1941	125 13 0	.
	Carried over ..	3,016 14 3	7,317 0 0

TABLE LXVII—CONTINUED.

Year	Estate's share only	Expenses	Income
		Rs. A. P.	Rs. A. P.
1941—*contd*	Brought forward ..	3,016 14 3	7,317 0 0
September 30th ..	To stationery in September ..	2 0 0	..
Ditto .	To labour and stores on works in September.	11 5 0	..
Ditto ..	To stores in September .	8 12 0	..
Ditto ..	To salary to lambardar for September	20 0 0	..
Ditto ..	To bazar manure during ½ year	378 14 0	..
October 23rd ..	To cycle allowance to muqaddam for September.	3 0 0	..
Ditto ..	To Railway freight on berseem culture from Lyallpur	1 12 0	..
November 10th ..	To stationery and stores in October	8 6 6	..
November 11th	To salary to Ilaqadar for October	20 0 0	..
November 20th .	By cash sale of rice straw	..	5 0 0
November 22nd .	To labour and stores on works in October	5 6 0	..
Ditto ..	To share of capital expenses for the year.	205 2 0	..
Ditto ..	To wages of beldars from April to September 30th.	252 0 0	..
Ditto	By produce received, wheat 528-34-6 at Rs. 3-5-0 average rate	..	1,851 0 0
Ditto	By produce Gram 525-30-8 at Rs. 2-12-0		1,446 7 0
Ditto	By wheat Bhusa 727 maunds at Rs. 0-6-0		272 10 0
Ditto	By Gur 37-18-14 at Rs. 1-12-0		65 9 0
Ditto	By linseed 0-5-0 at Rs. 2-8-0 ..	.	0 3 0
Ditto ..	By berseem [illegible] at Rs. [illegible] per maund		[illegible]
December [illegible] ..	By [illegible] at Rs. [illegible]		[illegible]
Ditto ..	By Rice straw from 140 acres at Rs. 5 per acre.	.	700 0 0
Ditto ..	By cotton received so far 256 maunds at Rs. 10-8-0	..	[illegible]
Ditto .	To balance [illegible] .	[illegible]	
	Total ..	[illegible]	[illegible]

TABLE LXVIII—CONCLUDED.

Water Analysis

RAKH MIANI

Registered No.	Site	Watertable	Water in the well	Date	Name of official taking samples
39	Chah Mona ..	4′—3″	18′—0″	19th March 1941.	Sardar Kirpal Singh, Analyst.
40	Chah Bawas .	13′—0″	..	20th March 1941.	Ditto
41	Station well .	16′—0″	25′—30′	Ditto	Ditto
42	Amir Chandwala .	7′—0″	50′	19th March 1941.	Ditto

Registered No.	39	40	41	42
Analytical data				
(All values in parts per 100,000) —				
Calcium Sulphate	*Nil*	*Nil*	*Nil*	*Nil*
Calcium Carbonate .	*Nil*	*Nil*	*Nil*	*Nil*
Calcium Bicarbonate (given as $CaCO_3$).	11·00	15·00	10·00	26·50
Calcium Chloride ..	*Nil*	*Nil*	*Nil*	*Nil*
Sodium Sulphate .	23·43	22·72	22·72	12·07
Sodium Carbonate ..	2·12	2·12	2·12	2·12
Sodium Bicarbonate (given as Na_2CO_3).	25·97	12·72	24·38	0·53
Sodium Chloride ..	9·36	8·77	5·85	5·85
Total ..	71·88	61·33	65·07	47·07
Total Solids Direct ..	73·2	63·0	63·5	45·8
Conductivity ..	1,000	870	850	700
pH value ..	8·38	8·10	8·20	8·18
Nitrate ..	..	..	..	..
Salt Index ..	—104	—7·25	—3·34	—17·14

TABLE LXIX

Makrach Nullah Flood Water Survey

SITE 1 Chitti Kothi Site 29th June 1941					SITE 2 Golpur Branch Site 20th June 1941				
Time of sampling	Conductivity	Total Solids parts per 100,000 of water	pH	Remarks	Time of sampling	Conductivity	Total Solids parts per 100,000 of water	pH	Remarks
12 noon	55,000	3,580·4	7·40		12-30 a.m.	70,000	4,290·0	7·50	
1 p.m.	20,000	1,201·6	7·73	Peak of the flood	1-30 ,,	30,000	1,763·6	7·70	
2 ,,	23,500	1,508·8	7·80		2-30 ,,	20,500	1,263·4	7·70	Peak of the flood.
3 ,,	35,000	1,900·2	7·78		3-30 ,,	22,500	1,325·2	7·62	
4 ,,	30,000	1,738·4	7·80		4-30 ,,	26,500	1,338·8	7·82	
5 ,,	35,000	1,985·8	7·90		5-30 ,,	34,000	1,962·8	7·81	
6 ,,	37,500	2,128·4	7·94		6-30 ,,	34,500	1,969·6	7·87	
					7-30 ,,	30,500	1,877·2	7·90	

TABLE LXXII

ANALYSIS OF SAMPLES OF WATERS FROM THE WAHNS (RUN OFF WATERS FROM THE SLOPES OF THE OUTER HILLS) DURING RAIN ON 6TH JULY 1941 AND 21ST AUGUST 1941

6th July 1941					*21st August 1941*				
Description of samples	Conductivity	Total Solids parts per 100,000 of water	pH	REMARKS	Description of samples	Conductivity	Total Solids parts per 100,000 of water	pH	REMARKS
Kohalan Wahn ..	..	..	..	Bottle broken in transit.	Kohalan Wahn	950	85·0	7·8	
Kachoola Wahn ..	1,700	149·2	7·5		Kachoola Wahn	915	85·1	7·8	
Dhaddar Wahn ..	1,150	93·6	7·6		Dhaddar Wahn	960	85·8	7·7	
Dangah Wahn ..	..	..	..	No water received in this Wahn.	Dangah Wahn ..	1,800	106·2	7·7	
Trimma Wahn ..	..	..	..	Ditto.	Trimma Wahn ..	6,200	470·0	7·5	

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